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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

EXPRESS HIGHWAY PLANNING IN METROPOLITAN AREAS

By Joseph Barnett,[1] M. ASCE

Synopsis

Perhaps, the need for free-flowing highway facilities to relieve traffic congestion in cities is evident, but attention is focused on this need by the superior facilities available on arterial routes in rural areas as compared with those in urban areas. The factors that influence the locations of arterial routes in cities and their effect on the city plan are discussed. The need for obtaining factual data is emphasized and the pattern of arterial routes developed in representative cities is shown. Brief comment is included on the subjects of by-passes versus radial routes, the need for flexibility, the use of existing streets, and the terminal problem. Relative merits of different types of expressways are presented. The need for, and description of, preliminary engineering reports to unite the numerous interested agencies are detailed, and an economic analysis of each project is suggested.

Introduction

Modern development of highway transportation has resulted in large differences between rural and urban roads, expecially in regard to the capacity of such roads to serve the traffic using them. Many cities are approached by numerous good roads of adequate capacity in rural and suburban areas; but in cities themselves, where the traffic reaches its highest volumes and where, therefore, there is the greatest justification for facilities permitting uninterrupted flow, the stops necessitated by street intersections increase in frequency, the average speed of travel decreases, and inconvenience and cost of motor vehicle operation rise sharply. The streets affected have undergone a progressive change in function from predominantly serving land to similarly predominantly serving new and mounting arterial traffic flows. Such improve-

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by August 1, 1946.

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ments as have been made have generally been designed to combine in one facility the service of both classes of traffic, the local and the arterial. As yet there have been few instances in which the desirable separate provision for arterial traffic has been attempted.

With the increase in motor vehicle travel, it was inevitable that this unbalanced provision of highway facilities in urban and rural areas would contribute to the decentralization of cities. Suburban housing developments naturally were near main radial routes out of the cities. The typical distribution of city population is created by many factors, and it is not the writer's intention to delve into them except to emphasize that the improvement of radial routes in the adjacent suburban and rural areas, without comparable improvements within the cities, was one of these factors. Most cities present the pattern of a central business or commercial district, an adjacent ring of dilapidated old buildings, generally old residences (either converted to commercial use or allowed to run down to substandard housing), a surrounding area of medium-grade or high-class housing of more than average density, and a creeping or fingerlike development—new, heterogeneous, and uncontrolled—along the radial routes leading out of the city.

In most instances the radial routes present a picture that is far from pretty. Unrestricted development along the roadsides has made entrances to cities via highways as unattractive as entrances via railroads. Instead of the industrial development that borders the railroads, the highways are fringed by a conglomeration of roadside businesses which, with their varied and blatant signs, vie with one another for the attention of travelers, detract from the sightliness of the city's gateway, and constitute a positive detriment to efficient highway transportation. To the credit of the railroads it may be stated that, except for grade crossings, which are gradually being improved in operation or eliminated, the roadbeds are as useful today for the movement of trains as when they were built. Contrariwise, on the average highway approaching a city, each new roadside business contributes its measure of interference, and the accumulation of such interferences steadily decreases the capacity of the road and its effectiveness in serving moving vehicles. A creeping obsolescence develops which is no fault of the highway engineer; it is rather the fault of antiquated laws and the pressure of real estate interests which make it difficult and, in some cases, impracticable to protect the traffic facility from roadside interference.

By-Passes

By-passes have frequently been constructed around cities on the theory that the adequacy of existing city streets would be improved by rerouting the cars of travelers who do not wish to stop in the city. Another justification for such by-passes, frequently cited, is the removal of heavy commerical vehicles from the main street, along which the through route usually is located. This expedient might offer not only the advantage of traffic relief but also the elimination of the noise and odor common to such commercial carriers. In many cases by-passes have proved to be useful adjuncts; and, indeed, they should be considered for many municipalities where traffic data indicate that

the need exists and where physical conditions permit. On the other hand, the fallacy of planning by-passes as a general solution of the city traffic problem has been glaringly revealed by the state-wide planning surveys, and other studies, which have shown that the bulk of the traffic approaching the larger cities is destined for points within the city itself and, for that reason, cannot be by-passed. Commonly, less than 10% of the traffic approaching cities of more than 300,000 population is destined for points beyond the urban area. By-passable traffic is about 20% in cities with populations of from 10,000 to 300,000, and only in communities of 2,500 or less does the by-passable traffic normally reach 50% of the total traffic.

Radial Highway Arteries Are the Greatest Need

Numerous fact-finding agencies have brought many highway transportation problems into focus. As a result, it has become increasingly clear that so much of the total traffic is urban in character that the need for allocation of funds for the improvement of highway facilities in metropolitan areas can no longer be ignored or relegated to a position of secondary importance. It has become clear, also, that traffic congestion in cities will be solved to the greatest extent feasible by the construction of facilities that permit the uninterrupted flow of traffic into and through the cities. The appalling number of traffic accidents is continually in the minds of highway planners; and, since most accidents occur at intersections in cities and a frequent type involves pedestrians, it is inevitable that the accident rate will be reduced by providing express highways on which crossings at grade are eliminated and pedestrians are not in close proximity to moving vehicles. As a consequence, highway planners have been giving preferential attention to plans for express highway projects that will provide solutions to the serious problems of city traffic congestion.

The mounting interest in urban arterial roads has been due to three general factors—(1) Availability of factual data; (2) necessity for accumulation of plans; and (3) the consciousness that planning for congestion relief has been inadequate.

(1) *Factual Data.*—Factual data regarding the movement and the origin and destination of motor vehicles in and approaching cities have been collected by various organizations such as state highway departments, other state agencies, county and city engineering organizations, planning commissions, and consultants. As a rule, these data leave much to be desired. Many cities have fairly good flow maps of traffic. Many cities have origin and destination data taken at an external cordon of stations on the main radial approach roads. Few cities have comprehensive origin and destination data that cover important intracity travel. However, regardless of the variation in the quantity or kind, such data almost always indicate that the most useful arterial road would be located radially with respect to the central business district. Since adequate traffic data reveal many more important needs, administrators supplied with such data have the comforting realization that they can proceed with plans with reasonable assurance that their judgment is based on, and backed by, facts.

Of all the varied data bearing on the planning of expressways in cities those showing the origin and destination of all traffic—through, suburban, and intracity—are most important in determining preliminary location of the expressway itself and in determining both the locations of access connections and the volumes of interchange traffic for which the expressways should be designed. The methods heretofore used in making comprehensive origin-destination studies are costly, laborious, and time consuming; and it is small wonder that few cities have undertaken to obtain data by such methods. The Public Roads Administration, Federal Works Agency, with the assistance and advice of experts of the Bureau of the Census, U. S. Department of Commerce, has developed a method for obtaining origin-destination data which promises to reduce cost, labor, and time sharply and at the same time enables the planner to arrive at more complete, accurate, and useful data. The method, roughly, follows the small-sample interview method used by public opinion polls except that the questions deal with travel facts such as "Where did you go yesterday, how, and for what purpose?" The questions are asked of occupants of a small percentage of selected sample dwellings in an area such as a census tract or part of tract, and the data obtained are expanded in proportion to the total dwellings of that area. Such questioning can be made by paid interviewers as rapidly as needed. The answers are transferred to coded punch cards from which the numerous factual data are summarized. A check on accuracy can be made by measuring traffic volumes at certain control points, such as river crossings and major intersections. As a result of the splendid cooperation of the officials of the cities and of the state highway departments, this method has been used, or is being used, in about thirty-eight metropolitan areas, and the number of cities is growing rapidly. It is felt that the "bugs" in the method have been eliminated.

Sometimes origin-destination data of traffic approaching a metropolitan area at a cordon of stations will be sufficient for locating an expressway. Such data will include by-passable traffic and traffic having a destination, or making a stop, in the city but the important intracity travel is missed. Origin-destination data at each station of a cordon are obtained by interviewing the drivers of a small percentage of vehicles leaving the city and expanding the results to the total traffic volume measured at the same time.

(*2*) *Shelf of Plans*.—No small measure of interest in urban arterials is due to the necessity for providing a shelf of plans for the construction of needed public works so that the construction industry could be prepared to provide jobs as soon as necessary. Quite naturally, industries are geared to city areas, and whenever there is unemployment, the necessity for jobs will be most severe in cities.

(*3*) *Relief of Congestion*.—The most important factor creating interest in urban arterial highways is the realization that only a small "dent" has been made in the solution of the traffic problems in cities of the United States. It is a law of nature (and in many cases a fortunate one) that people tend to forget past troubles. In many cities traffic conditions improved during the imposition of wartime driving limitations—to some extent because of the reduction in the number of vehicles. To a great extent, road users have forgotten the pre-

war miseries of congestion and will not be content to return to them. Highway engineers realize that there has been a huge unsatisfied travel demand; and they realize that it is imperative to meet that demand by planning the construction of free-flowing facilities that will solve the problem of congestion in urban areas. Widening, traffic-light control, one-way streets, rerouting, no parking in rush hours, and many other palliatives have been of great help, and will continue to be used; but the urban traffic problem will not be solved without generous provision for free-flowing facilities that will permit the uninterrupted flow of vehicles between points near origins and destinations.

Pattern of Arterial Routes

Discussions with state, county, and city officials regarding needed arterial highway facilities in urban areas inevitably invoke the statement that the particular city in question is different—it has special problems and cannot be considered a normal city. These statements generally are believed to be true; yet, when a comprehensive system of arterial routes in a city is laid out for long-term development, that city assumes a pattern which is unmistakably like those for almost all other cities of the same size. Adequate traffic data and the judgment of competent highway engineers inevitably lead to the conclusion that, to serve traffic best, arterial routes should be located so that the through routes outside the city are connected with the central business district. Practical considerations, such as the high cost of property in the central business district, require that the lines be relocated a little, perhaps several blocks, to the fringe areas that are so common in many cities. In the fringe areas, buildings have been allowed to deteriorate because no agency existed to control city development. These areas could not compete with the less expensive land surrounding them when additional housing was being constructed because their proximity to business made the land potentially valuable, although the time for conversion to business had not yet arrived. The houses in these areas could be rented as dwellings to low-income groups or to some types of businesses, and so could pay the carrying charges while the owners hoped that commercial expansion would absorb them. There usually has been no incentive to maintain the houses in good condition.

When the routes leading to the central business district are connected along these fringes, the pattern takes shape. In the larger cities, the procedure consists of the construction of a close-in circumferential route from which arterial roads to the outskirts of the city, and beyond, radiate in several directions. The pattern may twist, bulge, or be cut off on one or more sides; the inner circumferential route may be round, square, or elongated; the radial routes may be somewhat circuitous; and a large body of water or other topographic feature may block radial roads in some directions—nevertheless, the pattern is apparent. Fig. 1 shows the arterial highways, both existing and planned, in Cleveland, Ohio, one of the larger cities of the United States. The shaded area indicates the central business district.

In large metropolitan areas another characteristic part of the pattern is the outer circumferential route and sometimes an intermediate circumferential route. These routes are useful and, in some cities, very necessary facilities for

traffic between the outskirts or suburban areas and between points near different radial routes. They are not first-order priority projects, however, although sometimes considered first because of the relative ease with which the right of way can be acquired.

The pattern of arterial routes just outlined, even though forced by the circumstance of available right of way, is close to ideal in many respects. One factor that must be emphasized in developing a system of arterial routes is flexibility—flexibility in choice of routes presented to a driver and flexibility in fitting future construction to future needs. When the central business district is surrounded by an inner circumferential route, a driver has wide latitude in

Fig. 1.—Pattern of Arterial Highways in a Large Metropolitan Area

choice of routes. An individual driver is not destined for the central business district or any other area. Only in the mind of the planner or designer is he considered one drop in the stream of traffic headed for an area. The individual driver is headed for a point in an area and he should be able to choose a route that will take him on a free-flowing facility as close to the point of destination as feasible. A flexible system of expressways will reduce his time of travel to a minimum and permit travel with ease and safety—the fundamental reasons for making the improvements and providing the service the road user expects. In addition, if the driver arrives as close to his destination as feasible on a free-flowing artery, he is not obligated to travel local streets any more than absolutely necessary, thus relieving congestion in the heart of the city where relief is needed most.

Use of Existing Streets

The thoughts just expressed are the essence of the opposition to quite a different line of thinking by some planners. It has been proposed that arterial routes need not be developed as free-flowing facilities throughout their length, but rather that a type of design should be used to insure free movement on the radial routes entering a city where the expense of development is not significant. Such roads should be connected to one street, and preferably two or more existing surface streets, widened if feasible, through the central business district. On behalf of this plan, it is argued that the chosen route would not have to be on the edge of the central business district, but could be brought directly to the center of it and that, since such a large percentage of the traffic is destined for the central business district anyway, the purchase of expensive right of ways in or at the edge of the business district can be avoided and traffic given a wide choice of routes with normal street access at every block. One fallacy in such reasoning lies in the fact, just mentioned, that an individual vehicle is destined for a point in a district and use of surface streets should be reduced to a minimum. In addition, many drivers are not destined for the central business district but for an outlying district. They should not be encouraged to travel the local streets to go through the city or to reach another area in the city. Instead of relieving the area of most severe congestion, development along this plan will increase congestion because free-flowing facilities attract traffic from less desirable parallel routes and generate traffic that never existed before. To dump this traffic on to downtown surface streets already overcrowded would defeat a prime purpose of free-flowing facilities—the relief of congestion in the area of greatest congestion. In addition, there is no flexibility in such a plan for, if later observations indicate vital need for an expressway, the probability of being able to acquire sufficient right of way is nil and the door is closed to adequate development.

Surface streets are a necessary part of any comprehensive system of transportation for motor vehicles. They are the capillaries that carry the life blood of a city from the arteries to the limb extremities or final destinations, whether they are off-street or on-street parking areas, private or public garages, loading areas, or just street stops to drop off or pick up passengers or goods. Streets should be studied and regulated carefully to serve the area and abutting property in the desired manner. The interchanges between the free-flowing arterial road and local streets should be located and designed on the basis of traffic needs, effect on the arterial route, and effect on the surface distributors. The surface distributors may need adjustment to fit in with the over-all plan. These adjustments may consist of widening, conversion to one-way operation, changes in the traffic-light control cycles, or blocking cross streets to increase storage capacity—or any or all of these. Experience indicates that a complete plan is much more than "bulling" through a free-flowing type of arterial highway.

Stage Construction

A complete system of free-flowing arterial highways cannot be built overnight or even in a few years. The cost per mile of such facilities is so great that

a program of construction usually will have to be spread over several years to finance the improvement. To this end, routes to be improved should be divided into project sections that can be completed and connected to existing facilities to bring an immediate return in the form of road-user benefits on the investment made. Projects also can be constructed in progressive stages so that they can be used as facilities far superior to existing ones but not yet up to the ultimate design. For example, where cross streets do not carry large volumes of traffic, a section of expressway ultimately to be depressed can be constructed at grade. Other sections can be constructed with traffic on minor cross streets forbidden from crossing but permitted to make right turns to and

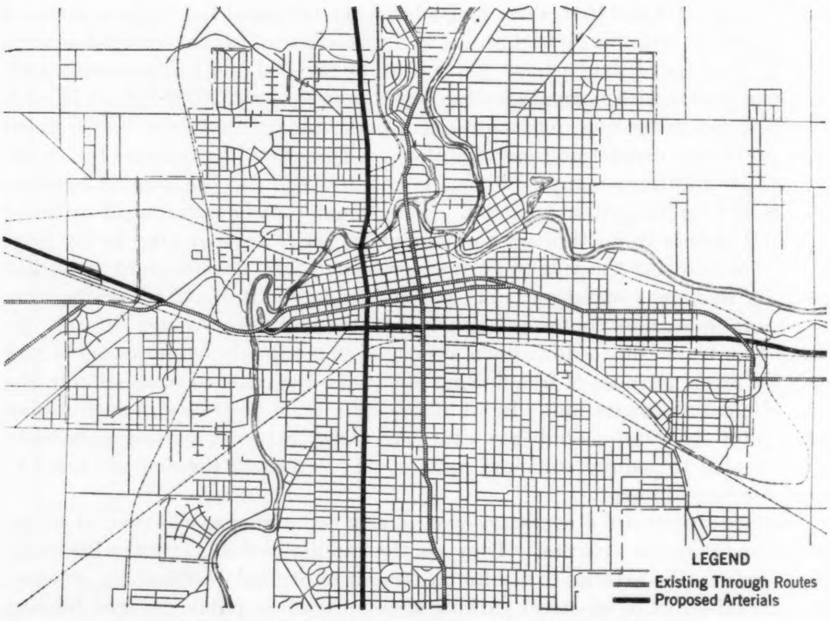

FIG. 2.—PATTERN OF ARTERIAL ROUTES IN A CITY OF MEDIUM SIZE

from the expressway. Only on important cross streets would traffic be allowed to cross. Parallel frontage roads sometimes can be provided to serve as terminals for minor cross streets and as protection for through traffic even where through traffic lanes are subject to traffic-signal control at important cross streets. A facility that permits almost uninterrupted flow of traffic can be provided by separating the grades at heavily traveled cross streets, allowing less important ones to cross at grade, and terminating all minor cross streets. Additional grade separations can be constructed as required. Where frontage roads are to be provided or existing streets serve as frontage roads, these roads may be used as an interim stage for through traffic until it becomes feasible to construct the through traffic artery. If the right of way for the ultimate development is available, numerous methods of stage construction can be employed.

Long-range predictions of future traffic requirements probably cannot be made with a high degree of precision. It becomes incumbent to develop, as well as possible, a broad general plan by which it will be possible to make such changes as may be found necessary to serve future traffic. The lines of a good

FIG. 3.—PATTERN OF AN ARTERIAL ROAD IN A SMALL CITY

general plan are likely to bend and bulge here and there, the traffic volumes to be served may vary to some degree, and the requirements for interchange may vary in number and volume; but the salient features of a well-conceived bold plan will not change appreciably.

The pattern of arterial routes just described, in which radial routes approach and are carried as close to the central business district as feasible, forming an inner ring, does not seem to materialize in cities of less than about 200,000. In such communities, two arterial roads at approximately right angles appear to be all that can be justified (see Fig. 2). This is a natural development since, generally, there are fewer radial routes and the central business district is smaller than in larger cities. With only two arterial roads in the smaller cities, the travel distance on surface streets to reach a destination may not be any greater than in larger cities with several arterial routes and an inner ring. In yet smaller cities, particularly where development is strung out along a railroad, a river, or an existing highway, one arterial road into and through the city generally is all that can be justified. The pattern in one such city is shown in Fig. 3. These are borderline cases and the origin and destination of local and through traffic should be examined to determine whether or not a by-pass is desirable.

Types of Expressways

Many types of streets and highways can be considered free-flowing facilities, and there are just as many conflicting opinions regarding the desirability of each type. The engineer can only discuss their merits and deficiencies.

Streets or highways at grade can be constructed with several well-known expedients so that traffic moves well on them with only moderate interruption. The mere designation of a street as a preference street by "stop" signs on all cross streets will aid materially. At heavily traveled cross streets, however, signs indicating preference are inadequate. Traffic-signal control, or officer control, during hours of peak load or all day, is necessary. Isolated traffic-signal controls may not aid free movement; they may be detriments to free movement, but are required to minimize accidents and to regulate traffic. A series of signals progressively controlling traffic may be considered an aid in freeing the movement of vehicles.

Interference from the sides of the road is not reduced by these expedients; only conflicts with cross traffic at intersections are affected. To free through traffic from roadside interference, it has been common, where width permits, to provide separate side roads for abutting property and to segregate through traffic between islands or barrier strips separating it from the frontage roads. These are the "boulevards" found on major thoroughfares in many cities. They can be developed for high capacity by permitting crossings at grade only at important cross streets and terminating minor cross streets at the frontage roads. A facility of this type is shown in Fig. 4. Still greater capacity and freedom for through traffic can be developed by providing a divider between opposing traffic and by occasional grade separations.

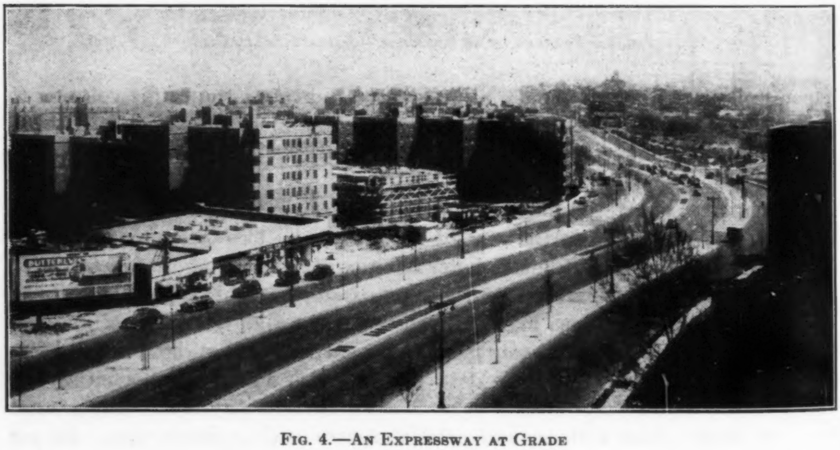

Fig. 4.—An Expressway at Grade

A surface expressway, although it is a tremendous improvement over any thoroughfare with all streets crossing at grade and is capable of handling large volumes of through traffic, is not truly a free-flowing facility. Traffic-signal controls are required for the occasional street crossing at grade and for the more

frequent pedestrian crossings at grade. Structures that require pedestrians to walk up or down to bridges or underpasses are not used except under severe compulsion, can become nuisances, and often do not justify their cost. Pedestrian facilities will be used without strict control only when entrance to them is easier than crossing the expressway at grade—such as a footbridge at the general level of adjoining streets crossing over a depressed expressway.

The expressway type that has proved the most desirable in developed urban areas is that in which the express facility is at or below the general level of the surrounding area, no direct access with abutting property is provided, and all crossings at grade are eliminated. An example is shown in Fig. 5. In addition

Fig. 5.—A Depressed Expressway

to providing free movement for all through traffic, the depressed type of expressway does not present a visual physical barrier between the areas adjacent to the expressway, and it is flexible as regards future requirements of cross traffic in that additional structures across the expressway can be provided readily, with little or no interference with through traffic during construction.

Control of Access

The control or limitation of access is necessary to the proper functioning of a free-flowing expressway. Sometimes (particularly on new facilities) this can be obtained when purchasing the right of way, provided the laws of the state permit. Controlling the right of access is not a simple matter. The rights of citizens to enter a public highway are rooted deep in fundamental common law

and are not easily withdrawn.[3] The surest safeguards appear to be the purchase of these rights from the owners of adjacent property and the provision of frontage roads along both sides of the expressway to serve abutting property. Sometimes these lanes are newly constructed. Frequently, one or both are existing streets. Where the line of the expressway is generally parallel to an existing thoroughfare in a grid pattern of streets so typical of cities in the United States, it is preferable to acquire right of way between the streets rather than to widen one of them. Maintenance of, and changes in, public utilities in cities—particularly underground lines—are costly and annoying. Changes in utilities parallel to the expressway can be reduced to a minimum by locating the expressway between existing streets, which continue to act as frontage roads and as media for utilities. The maintenance of traffic in cities during construction also is important. By locating the proposed expressway between existing streets, the latter can function as before with interference resulting from construction operations limited largely to work on cross streets.

Elevated Structures

The elevated structure type of expressway (see Fig. 6) is the most costly of all, as regards construction. There is a prevailing impression that such expressways save right-of-way costs, and particularly that they can be located on existing streets without the acquisition of additional right of ways. This impression is correct to a limited extent only. Elevated structures can be constructed on right of ways barely wide enough for the structure proper, but this rarely is desirable. Existing streets have distinct functions in serving abutting property and in accommodating moving traffic, even though limited in capacity by the cross streets. Consequently, the placing of columns in the street for the support of an elevated structure seriously restricts the usefulness of the street. The structure itself blocks off light, creates a potential nuisance area below, and seriously damages property on both sides. It should be realized that, where elevated structures can be justified, the expected traffic volume usually is high enough to justify a four-lane or a six-lane facility which requires a width nearly that of the average city street.

Undesirable features of an elevated structure are avoided to some extent by increasing the width of right of way to accommodate surface frontage roads clear of the structure supports, thus releasing the area under the structure for parking or other businesses. The increased width generally is needed, in any case, at intervals, to provide space for ramps. The use of an area under an elevated structure for business purposes, say, retail or warehousing ventures, is desirable in that revenue is forthcoming and the policing and maintenance of the area is transferred to the owners of the establishments although the added fire hazard must be considered. By the time that the various refinements are worked out and a sufficient width is obtained to minimize the damage to abutting property, the required right of way probably is sufficient for the development of a depressed expressway.

There are two major advantages to elevated expressways aside from pro-

[3] "Public Control of Highway Access and Roadside Development," by David R. Levin, Public Roads Administration, Washington, D. C., 1943.

viding space below the structure for some useful prupose: (*a*) Advantage to local cross traffic; and (*b*) maintenance of utilities.

(*a*) Nearly all streets can cross under the expressway without interference with through traffic—indeed, with less interference than existed prior to construction since the major traffic stream will use the overhead structure. An occasional street will be blocked off by a ramp.

(*b*) Utilities crossing the expressway are practically undisturbed by elevated structures. In some cities changes to utilities may be a major item of cost.

Sometimes topography will make elevated structures a necessity regardless of other considerations. If topography is rolling, a desirable control gradient

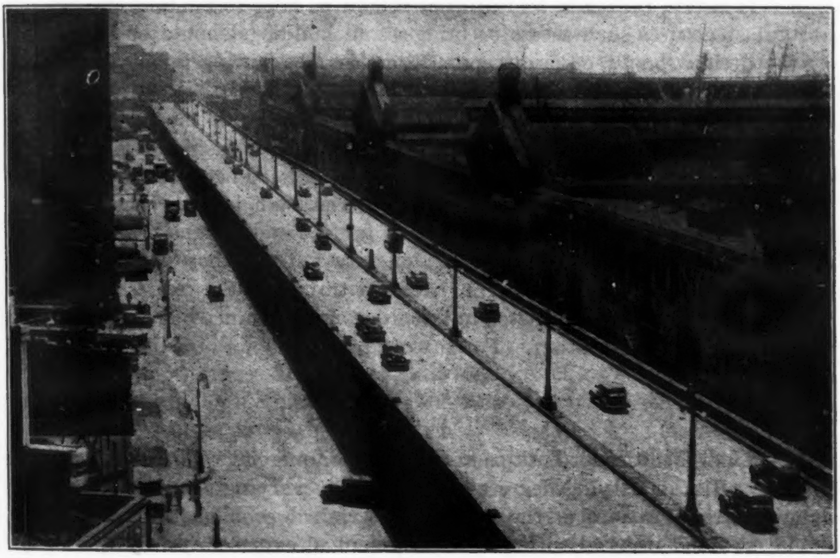

Fig. 6.—An Expressway on an Elevated Structure

of 3% or less may be feasible only where the expressway is in a depressed section crossing high ground and on an elevated structure crossing low ground. Low gradients are desirable, particularly for trucks which may slow down and reduce the capacity of the facility on even moderate gradients. On long steep, or even moderate, gradients, trucks are reduced to crawl speeds, seriously impeding following vehicles. This condition has been alleviated on existing two-lane roads by providing an additional distinctive lane for slow-moving vehicles upgrade. Truck drivers show a high degree of compliance. Where long gradients cannot be avoided on urban expressways, the use of such added lanes should be considered to avoid a reduction in capacity of the upgrade pavement.

Principles of Design

The design of an expressway, of each interchange and of each focal area such as a terminal, should be examined in the light of the principles that follow: (a)

The facility must fit traffic needs; (b) traffic operation on the completed facility must be free flowing and safe; (c) the path of the individual driver (at least that in a major traffic movement) should be as direct and simple as feasible and the required actions on his part be natural; (d) the operation should be flexible so that a driver will have a wide choice of destinations; (e) the facility should be flexible so that it can fit into any reasonable future change in the traffic pattern and so that it can be connected to important traffic facilities that may be provided in the future; (f) the facility should be integrated with the city plan; (g) there should be a wide distribution to the street system; (h) stage construction and development of short usable sections should be practicable; (i) it should be possible to maintain traffic adequately; (j) disruption of existing transit facilities, railroads, and utilities should be kept to a minimum; (k) important structures such as major bridges and buildings should be left intact; (l) the design should envision connecting the free-flowing facility to similar facilities beyond the immediate area of the problem; (m) the cost must not be out of line with the service rendered; (n) maintenance of the facility will be nominal; and (o) the completed facility should be pleasing in appearance.

The critical examination of interchanges is suggested because they are much more important in expressways in urban areas than in any other highway facility. Interchanges that are improperly located and designed can aggravate rather than relieve traffic congestion on city streets; they can affect their desired pattern adversely and can back up traffic on the expressway. The foregoing principles are not listed in the order of importance but to group those that are related to each other. For any one problem not all the listed principles may be applicable.

Terminal Facilities

Terminal facilities for both passenger and commercial vehicles are available in many cities in varying degrees and for widely varying charges. There are numerous ramifications of the important terminal problem; but its solution should go hand in hand with the development of expressways. Expressways in cities generate a considerable volume of traffic which tends to aggravate congestion in terminal areas if steps are not taken to provide for it. To solve the terminal problem, it is desirable that all interested agencies, particularly local public and private agencies, attack it broadly in the form of zoning control and the development of off-street parking and loading facilities by private and public interests. In many cities existing parking lots cannot be depended upon for future use. Their land value is so high that improvement by construction of commercial buildings should be expected as the city expands. An interesting development in some cities is the multiple use of such buildings for stores where retail frontage is valuable, for parking on the lower floors easily accessible to the streets by ramps, and for commercial businesses on the upper floors to which elevator service is convenient.

The problem of truck loading will be solved only by a bold approach. There appears to be no less justification for loading trucks inside business establishments than for loading freight cars on private sidings. The truck terminal, whether developed privately or as a union terminal, lessens street congestion

in that it decreases the truck mileage in congested areas and may reduce the size of trucks operating therein. However, major improvement will require loading-dock space inside the building line of the ultimate destination of the goods carried. In some locations, the result of limiting or eliminating first-floor street frontage will not be an undue hardship; and, at other locations, passages to the rear of the property or ramps to upper floor docks will be justified. Zoning to require, or at least to encourage, development of dock space inside the building line appears justified.

Planning Agencies

Orderly procedure in express highway planning may change for different cities because of variations in the distribution of planning functions. The planning of express highways logically can be considered the function of the state highway department, the city engineer, the county engineer, the city plan commission, or the highway committee of the chamber of commerce—or any or all of these. Some cities have presented a spectacle of so much planning that what should have been a healthy rivalry resulted in a jam which prevented progress. In such cases the efforts of a coordinator become necessary to induce the various agencies to bury minority differences and to agree on a plan acceptable to all. These problems are the concern of all and it is gratifying to note, in many cities, a healthy cooperation between the several agencies interested in the cities' welfare.

Major highways in cities are the main arteries of transportation for people and goods; and, if they enable vehicles to travel only at low average speeds, the city suffers in both increased cost and time for all functions depending on transportation. If the major highways are developed as free-flowing expressways, the tempo of transportation in the city can be increased with favorable results for all. The writer cannot agree with visionary enthusiasts who believe that freeing transportation will of itself rehabilitate a city, although it can be of material aid. The forces of decentralization are too great to be stopped entirely, but the proper location and development of expressways can assist in the establishment of self-contained stable neighborhoods and in the stabilization of trade and values in the principal or central business district.

The self-contained neighborhood appears to be one objective on which most city planners agree. A neighborhood should be roughly from a half mile square to one mile square, and should contain, in addition to housing, all the essentials of living, such as schools, places of worship, community and recreation facilities, and the everyday stores—the butcher, the baker, and the modern version of the candlestick maker, the five-and-ten-cent store. The central business district would retain the larger establishments such as the department stores, clothing establishments, commercial and industrial offices, and banks. Properly located expressways can serve the triple function of acting as dividers between neighborhoods or between a neighborhood and an area of a different type, as transportation mediums for highway traffic between neighborhoods and the central business district, and as transportation arteries for through highway traffic. When travel to the central business district takes but a short time and the road users have convenient terminal locations for parking, loading,

and unloading, the tendency for the larger establishments to move from the central area to the neighborhoods is likely to be retarded and the central business district stabilized. A detailed discussion of city planning as a whole is beyond the scope of this paper; but the highway engineer must appreciate that, in the location and design of urban arterial facilities, he is participating in city planning. Accordingly, it is essential that he cooperate intimately with all other agencies interested in the planning, development, operation, control, and general welfare of the city.

Preliminary Engineering Report

One of the most useful mediums to unite all interested agencies in the consummation and approval of a plan for an expressway is the preliminary engineering report. Such a report may cover one or several projects. A properly prepared report has several useful functions. First and foremost, it serves the designer himself by permitting him to examine the proposed solution objectively and to assure himself that the proposed facility will solve the transportation problem, that it fits in with the numerous related projects, that the design is feasible, and that the broad picture is complete. A designer may be so engrossed in detail that he loses the over-all picture—loose ends are not garnered and impracticable features creep in.

The preliminary engineering report helps to acquaint all agencies with the problem and its solution, to develop constructive criticism, to unearth errors in so far as it is possible to do so, to fit the project into the over-all city plan, and to focus the attention of fiscal authorities on funds needed for advancing the project from preliminary engineering status to right-of-way acquisition and final construction.

The cost of a preliminary engineering report almost always is manifoldly justified. Free-flowing express facilities through cities are complicated. Right of way is costly, numerous utilities are affected, and alternate locations or even slight adjustments in the line and the access connections might have serious consequences. Unlike those for rural highways, the construction plans for urban expressways may be materially affected by such adjustments and changes in the plans are costly and time consuming. It is desirable, therefore, that the salient geometric features of the design, arrangement of roads, alinement, width, accesses, walls, bridges, and other design features and standards be "tied down" before construction plans are begun. The preliminary engineering report can be used for such purpose. When the approval of all interested agencies is at hand, the detailed design can proceed with assurance that no radical changes will be made.

Preliminary engineering reports vary considerably with each project or group of projects and with different administrators. In some cases a line has previously been established definitely, and the report may show the results of location surveys and plans in considerable detail. In other cases the preliminary engineering report may present no more details than the general location and justification usually found in a city planning report. In such cases the report should be considered one of a series, the later reports going into greater detail before construction plans progress very far. In general a preliminary

engineering report should include five broad phases: (I) Transportation, (II) Design loads, (III) Solution, (IV) Estimate of cost, and (V) Justification.

(*I*) *The Transportation Problem.*—The report should indicate the general transportation problem to be solved. To this end it should show the traffic and transportation situation. Where does traffic originate and where it it destined? What traffic flow exists as a result of traffic pressures and what traffic flow would exist under unrestricted conditions? What delays are encountered? How does traffic, as it flows, affect other functions of the city? How are industry, trade, and commerce influenced? How is mass transportation concerned and to what extent will the proposed facility serve it? How are pedestrians affected? What is done with vehicles when they reach their downtown destinations? All such questions should be answered in the preliminary engineering report. Existing conditions should be depicted by topographic maps and aerial views, both oblique and mosaic. Maps and graphs showing land use, social trends, and other data are useful in showing the problems that transportation in the city is facing. Land value and improvement maps are invaluable in analyzing the probable cost of right of ways in choosing locations. Utilities should be indicated, particularly those underground.

(*II*) *Design Loads.*—The report should indicate the traffic volumes for which the expressway is to be designed—either in that part of the report dealing with the problem or in that part showing the proposed solution; but traffic volumes should be shown somewhere. The design traffic volumes are the "loads" to be accommodated. There is no more logic in designing a highway facility without these loads than there is in designing a bridge structure without knowing the weights and distribution of the wheel loads it is to carry. All too frequently, the available traffic data are not sufficient to determine the probable traffic volumes that will move on the expressway and the probable number of vehicles that will make the several turns at each point of interchange. In such cases the data should first be obtained; but, where this is not feasible, the probable volumes should be determined by judgment (using the best data available) and thus recorded as a part of the preliminary design data. This procedure is superior to one in which only the number of traffic lanes is decided by the judgment of the same engineers.

(*III*) *Solution.*—The report should give the proposed solution, including a preliminary design plan and profile, which may be drawn to a scale of about 1 in. = 400 ft. They should show the edges of all pavements, including frontage roads, access connections, and adjacent streets. Structures and walls should be indicated. Landmarks and adjacent areas affected by the construction also should be shown. The alternate locations studied in arriving at the final location, as well as the alternate designs tried at each intersection, should be marked; and an account given of the reasons for choosing the proposed design. The reports will pass through many hands, and a presentation of alternate ideas will help reviewing and affiliated agencies to avoid repetition of the same process of trial and elimination.

A preliminary engineering report should be prepared in a manner clearly understandable by the many officials and other individuals who are not engi-

neers and to whom a blueprint is a mysterious document. Some parts of the design, particularly directional types of interchanges, are difficult to visualize in three dimensions. Such locations should be shown in perspective delineations which give clear pictures of what is intended. In some cases, models are made although they generally are expensive, require a long time to construct, and frequently do not show as large an area as one or more perspective delineations. These are made in an amazingly short time by men who generally are not engineers but who have the peculiar "knack" or gift required for that type of work. Aerial pictures, with the proposed facility drawn in, also have been advantageous. General pictures of what is intended are very useful when it becomes necessary to acquaint the public at large with the project. Newspapers welcome material of this character.

(*IV*) *Estimate of Cost.*—The preliminary engineering report should include a reasonably accurate estimate of the cost. Such cost should be broken down into the usual subdivisions of right of way, construction, engineering, and contingencies. In addition, it should be broken down further into sections which, preferably, would be usable upon completion. Expressways in cities take time to design, finance, and construct, and the administrator should have the cost estimate in such flexible form that it can be fitted into an ever-changing fiscal program. To this end the report should show the manner in which sections of the expressway can be constructed in stages and the cost of each stage.

(*V*) *Justification.*—The preliminary engineering report should describe fully the reasons justifying the expenditures proposed for the construction of the project—in particular the transportation problem, which it is intended to solve by the expressway, and how the solution will be accomplished. Frequently, reports discuss congestion and interference with industrial or commercial functions, and sometimes accident frequency, and follow such discussion by a description of the relief that will be afforded by the new facility and how it will be obtained. The effect of the expressway on the city plan and on the development and stabilization of important areas might well be included. These are worthwhile reasons for prosecuting the project, and the benefits sometimes are sufficiently evident to be enough justification; but, for a realistic businesslike approach, the report should include an economic analysis of the project.

An important part of an economic analysis is the balance sheet, showing on the one hand the annual cost and on the other the annual benefits. Since this paper deals with expressways, a treatment of benefits due to direct land service use can be omitted without appreciable error, in order to deal entirely with benefits to road users. The cost of an expressway in a city is a capital expenditure and should be sanctioned only after an analysis has been made to determine what benefits will accrue to road users. Road-user benefits are not the only returns, as has been noted. Intangible benefits, such as ease and comfort of driving and facility and convenience of travel, and indirect benefits, such as effects on city stabilization or development, may outweigh benefits to road users in some cases; but an economic analysis that compares road-user benefits with annual costs should be made. This relationship is the guide that administrators need in determining economic feasibility of a project or in com-

paring alternate solutions to a transportation problem, for it indicates in businessmen's terms the road service to be provided by the proposed expenditure of funds.

The annual cost of a project may be assumed to be the sum of the annual cost of financing plus the annual cost of maintenance and operation. The subject of finance is beyond the scope of this paper, but it is well to state that the annual cost of financing is the same for any one rate of interest and any one period of time of amortization, regardless of the method of financing adopted. The annual road-user benefits are the saving in the cost of vehicle operation, the saving in time, and the saving resulting from accident reduction. Of course, there are some differences in observations (and in their interpretation) for the cost of vehicle operation on various types of highway facilities. There are differences of opinion regarding the factors to be used in calculating the value of saving in time although there is general agreement that time is valuable even for pleasure-bent passenger vehicles. If factors are inaccurate to a great degree, the resulting analysis can be misleading; but, if available factors are reasonably accurate, the economic justification for a project should be developed and included in the preliminary engineering report. Such analyses are particularly useful in comparing two or more solutions for the same problem or in assigning priority ratings to several projects.

Considering the large number of vehicles involved, the savings in vehicle-operation costs effected by the elimination of "stop-and-go" driving and other speed changes, and the time saved in traveling steadily even if not at high speed, expressways in cities show high returns on the investment, well beyond comparable returns on most other types of highway and street improvements. The demonstration of this fact to the driving public will give administrative officials the courage to satisfy demands for expressways. Such demands are certain to increase in volume and intensity as the inevitable congestion in cities returns; as those responsible for the welfare of a city look for every aid in its rehabilitation, stabilization, or further progress; and as the driving public becomes acquainted with the many desirable features of travel on free-flowing protected highway facilities.

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PAPERS

TORSION IN STEEL SPANDREL GIRDERS

BY J. E. LOTHERS,[1] M. ASCE

SYNOPSIS

The computation of torsional stresses in rolled steel sections is not new. The application has been confined, however, to eccentric loads where the torque, or torsion moment, could be obtained by multiplying the load by the eccentricity. The case of hanger bolts suspended from a steel girder flange and supporting an intermediate stair landing might be cited as an example. A steel spandrel girder with a floor beam framed into its web presents a torsion problem of a different type. The deflecting floor beam twists the spandrel and a torque is introduced—the computation of which is analogous to that of the bending moment brought to a column by a rigidly connected beam. This torque has been neglected for the following reasons: (1) No simple method of computation has been advanced; and (2) it has not been considered of sufficient importance to justify an extended analysis.

The purpose of this paper is to propose simple formulas for the solution of the aforementioned problem and to justify them by the method of moment distribution. Simple empirical formulas are also proposed for computing approximate values of the torsional constant K for the design of rolled steel beams which involves only beam constants found in the ordinary steel handbook.

INTRODUCTION AND RESTATEMENT OF PRINCIPLES

Notation.—The letter symbols in this paper are defined where they first appear, in the text or in illustrations, and are assembled alphabetically, for convenience of reference in the Appendix. Discussers are requested to adapt their notation to that of the paper.

Formulas Applicable to Cylindrical Beams.—In the ordinary textbook on mechanics of materials, torsion is applied to cylindrical shafts, that section being the most economical disposition of material to resist torsion. In other

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by August 1, 1946.

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words, if a beam is to be designed, the chief function of which is to resist torsion, a circular section is chosen. The following two equations apply:

$$s = \frac{T\,r}{J} \qquad (1a)$$

and

$$T = \frac{J\,G\,\theta}{L} \qquad (1b)$$

in which s is the torsional shearing stress at the surface of a cylindrical shaft or beam; T is the torsional moment or torque; r is the radius of the shaft or cylindrical beam; J is the polar moment of inertia of the section; G is the shearing modulus of elasticity (usually assumed to be 0.4 E); θ is the total angle of twist; and L is the length of the shaft or cylindrical beam over which θ has been accumulated.

Eqs. 1 may be used for noncircular sections, such as I-beams or channels, subjected to torsion provided that proper values are substituted for r and J. If the thickness of that part of the noncircular section for which the torsional shear is desired is substituted for r and if the torsional constant K for the section is used instead of J, a close approximation of the desired result will be obtained by Eq. 1a.

The Torsional Constant K.—The torsional constant K may be defined as the measure of the torsional rigidity and twisting deflections of the beam.[2] For circular sections, J and K are identical. For noncircular sections, K is always less than J, but there is no direct relationship between the two. The use of K in connection with noncircular sections is analogous to that of J for circular sections. Therefore, the following equation holds:

$$K = \frac{T\,L}{G\,\theta} \qquad (2)$$

The value of K for any given beam may be found experimentally by observing simultaneous values of T and θ and substituting them in Eq. 2. For I-beams and H-beams it may also be found, for sloping flanges, by:[3]

$$K = \frac{b - t_w}{6}(m + n)(m^2 + n^2) + \frac{2}{3}t_w\,m^3 + \frac{1}{3}(d - 2\,m)\,t^3_w + 2\,\alpha\,D^4 - 4\,U_s\,n^4 \qquad (3a)$$

and, for parallel-sided flanges—

$$K = \frac{2}{3}b\,n^3 + \frac{1}{3}(d - 2\,n)\,t^3_w + 2\,\alpha\,D^4 - 0.42016\,n^4 \qquad (3b)$$

in which α is a factor that depends on two ratios, $\frac{t_w}{m}$ and $\frac{R}{m}$; and U is a factor depending on the $\frac{b}{n}$-ratio, with subscripts L and s to denote "large" and

[2] "Structural Beams in Torsion," by Inge Lyse and Bruce G. Johnston, *Transactions*, ASCE, Vol. 101, 1936, p. 857.

[3] *Ibid.*, Eqs. 20 and 21, pp. 864–865.

"small" ends, respectively, of the flange. Symbol D is the diameter of the inscribed circle drawn tangent to the top of the flange and to the two adjacent fillets, and it may be computed or determined graphically by a large-scale layout of the beam.[3] The significance of the remaining symbols is shown in Fig. 1.

According to Saint Venant[2] and L. B. Tuckerman[4] the value of the torsional constant K for a structural steel section may be found approximately by dividing the section into three rectangular parts and taking one third of the long dimension times the cube of the short dimension for each part. By summation:

$$K = \frac{1}{3}\,d t^3_w + \frac{2}{3}\,(b - t_w)\,t^3_f \qquad (4)$$

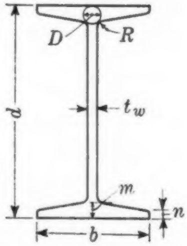

FIG. 1.—SYMBOLS

in which t_f is the mean flange thickness. Finding that Eq. 4 gave results that are too small, Mr. Tuckerman multiplied the right-hand side by a factor based on tests. In using Eq. 3b, Inge Lyse and Bruce G. Johnston,[2] Members, ASCE, applied corrections to take care of end effects, the stiffening effect of fillets, and the added rigidity due to the connection of the flange and web. In a like manner, Eq. 3a provides for the K-values of sloping-flange sections.

In an effort to arrive at simple formulas that would give practicable, approximate values for K and which would involve only those beam constants that are given in the ordinary steel handbook, the writer transformed Eq. 4 into the form,

$$K = c\,d t^3_w + 2\,c\,(b - t_w)\,t^3_f \qquad (5)$$

in which c is an empirical constant to be evaluated by substituting known values for the other constants in Eq. 5. Rolled steel beams were divided into three groups—(1) The square column or H-sections with parallel-sided flanges, (2) the remaining wide-flange sections, and (3) the American standard I-beams. A value for c was computed for each group, resulting in a formula for each group.

To arrive at reasonably representative values for c, beam constants for twenty-seven, well-distributed sections for each group were substituted in Eq. 5. The values substituted for K were taken from the "Bethlehem Manual of Steel Construction"[5] and were originally computed by Eqs. 3. The average value of c computed for each of the three groups resulted in the following equations:

For square column sections or H-beams with parallel-sided flanges—

$$K = 0.36\,d t^3_w + 0.72\,(b - t_w)\,t^3_f \qquad (6a)$$

for the other, nonsquare, wide-flanged beams, including both parallel-sided and sloping flanges—

$$K = 0.39\,d t^3_w + 0.78\,(b - t_w)\,t^3_f \qquad (6b)$$

[4] "Tests of I-Beams in Torsion," by L. B. Tuckerman, *Engineering News-Record*, Vol. 93, 1924, p. 882.

[5] "Bethlehem Manual of Steel Construction," Bethlehem Steel Co., Bethlehem, Pa., 1934, p. 279.

and, for American standard I-beams—

$$K = 0.43\, d t^3_w + 0.86\,(b - t_w)\, t^3_f \ldots\ldots\ldots\ldots\ldots (6c)$$

Torsional Shear Equations for Rolled Steel Sections.—For H-beams and I-beams, the following equations were suggested by Messrs. Lyse and Johnston:[6]

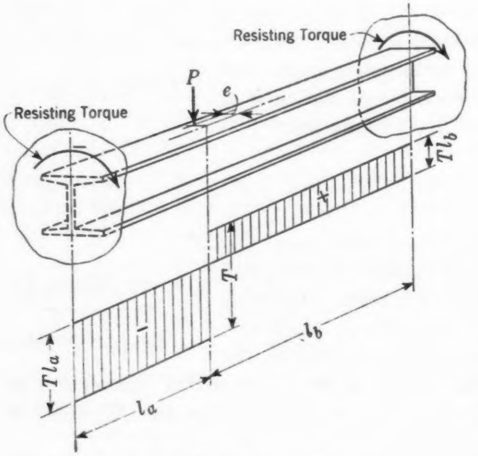

Fig. 2.—Distribution of Torque

$$s_w = \frac{T\,(t_w + 0.3\,R)}{K} \ldots (7a)$$

and

$$s_f = \frac{T\,(n + 0.3\,R)}{K} \ldots (7b)$$

in which s_w and s_f are the torsional shearing stresses in the web and the flange, respectively.

Distribution of Torsion in a Beam.—A beam with a torsional load applied at some point along its length is subjected to a torsion or torque, T, which is distributed between the two ends of the beam in inverse proportion to the lengths of the two legs (see Figs. 2 and 3). This follows from the fact that the distribution of torque is pro-

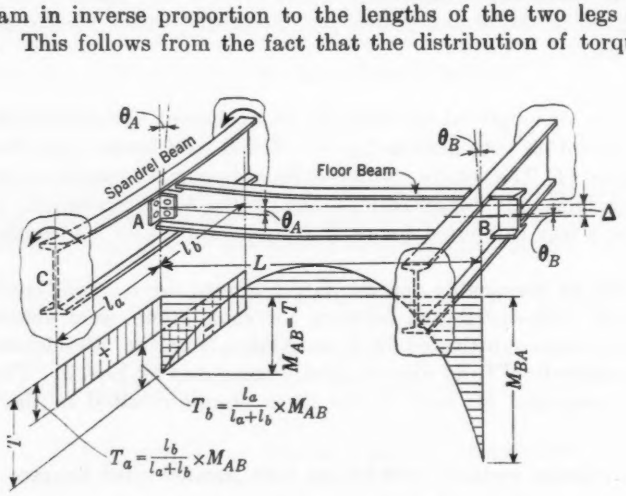

Fig. 3.—Distribution of Moment and Torsion

portional to twisting stiffness which, in turn, is inversely proportional to length. The following equations apply and should be studied in conjunction

[6] "Structural Beams in Torsion," by Inge Lyse and Bruce G. Johnston, *Transactions*, ASCE, Vol. 101, 1936, Eqs. 27 and 28, p. 867.

with Fig. 2:

$$T = P\,e \quad \text{(8a)}$$

$$T_a = \frac{T\,l_b}{l_a + l_b} \quad \text{(8b)}$$

and

$$T_b = \frac{T\,l_a}{l_a + l_b} \quad \text{(8c)}$$

Torsion in Steel Spandrel Beams

Approach to the Problem.—In attacking the problem of torsion in spandrel beams several questions arise:

(1) How much moment can be developed at the end of a floor beam that frames into the web of a spandrel? In other words, what is the relative stiffness of the two beams, the one being in flexure and the other in torsion?

(2) How shall this moment be distributed among the two legs of the spandrel and the end of the floor beam?

(3) What assumptions shall be made as to the rigidity of end connections?

Relative Stiffness.—A beam is not nearly so stiff in torsion as it is in flexure. Therefore, since the end moment of a beam is proportional to its restraint, a relatively small moment is developed at the spandrel end of a floor beam. As shown in Fig. 3, the moment M_{AB} at the end of the floor beam must equal $T = T_a + T_b$. Also, assuming sufficient rigidity in the connection between the end of the floor beam and the spandrel web, the rotation θ_A of the end of the floor beam equals the angle of twist of the spandrel. From Eq. 3, $T_a = \frac{G\,K\,\theta_A}{l_a}$ and $T_b = \frac{G\,K\,\theta_A}{l_b}$. Therefore:

$$T = \frac{G\,K\,\theta_A\,(l_a + l_b)}{l_a\,l_b} \quad \text{(9)}$$

The slope-deflection equation for the moment at end A of the floor beam is:

$$M_{AB} = \frac{4\,E\,I\,\theta_A}{L} + \frac{2\,E\,I\,\theta_B}{L} - \frac{6\,E\,I\,\Delta}{L^2} - (\text{FEM})_{AB} \quad \text{(10)}$$

Comparing Eqs. 9 and 10, it is evident that the same θ_A which gives rise to a torque of $\frac{G\,K\,\theta_A\,(l_a + l_b)}{l_a\,l_b}$ in the spandrel contributes $\frac{4\,E\,I\,\theta_A}{L}$ to the end moment M_{AB} of the floor beam. Hence, the relative stiffness or flexural stiffness of the floor beam becomes:

$$S_F = \frac{4\,E\,I}{L} \quad \text{(11a)}$$

and the torsional stiffness of the spandrel girder is:

$$S_T = \frac{G\,K}{l_a} + \frac{G\,K}{l_b} = \frac{G\,K\,(l_a + l_b)}{l_a\,l_b} \quad \text{(11b)}$$

Moment M_{AB}.—The entire problem hinges on the moment M_{AB} at the spandrel end of the floor beam. This moment must be absorbed by the

spandrel girder; it produces the twist or torsion in the girder. Therefore, if a formula for M_{AB} can be found, the major part of the problem is solved. The following remarks are pertinent to this moment:

(*a*) The sum of the torques in the two legs of the spandrel must equal the moment M_{AB} (see Fig. 3).

(*b*) In view of the relatively low value of the torsional stiffness of the spandrel as compared to the flexural stiffness of the floor beam, the end moment M_{AB} of the latter is small and, by the same token, the ordinary standard connection between the two may be considered rigid. Any error in this assumption is on the conservative or safe side.

(*c*) The moment M_{AB} depends greatly upon the degree of restraint at the inner end or end B of the floor beam. Three different conditions of restraint will be assumed and a formula for M_{AB} given for each case.

Case I.—The inner end or end B of the floor beam is assumed to be fixed:

$$M_{AB} = \frac{S_T}{S_T + S_F} \times (\mathrm{FEM})_{\mathrm{AB}} \qquad (12a)$$

Case II.—The inner end or end B of the floor beam is assumed to be free:

$$M_{AB} = \frac{2\,S_T}{S_T + S_F} \times (\mathrm{FEM})_{\mathrm{AB}} \qquad (12b)$$

Case III.—The inner end or end B of the floor beam is assumed to be 50% fixed:

$$M_{AB} = \frac{3\,S_T}{2\,(S_T + S_F)} \times (\mathrm{FEM})_{\mathrm{AB}} \qquad (12c)$$

In Eqs. 12, S_T represents the torsional stiffness of that part of the spandrel girder which resists the moment M_{AB}, and S_F signifies flexural stiffness and refers to the floor beam. The abbreviation $(\mathrm{FEM})_{\mathrm{AB}}$ denotes the fixed-end moment at end A of the floor beam. (See Eqs. 11 for values of S_F and S_T to be used in Eqs. 12.)

Eq. 12*a* holds for any type of loading on the floor beam so long as end B is fixed. Eqs. 12*b* and 12*c* for cases II and III, however, hold only for symmetrical loading on the floor beam AB. For unsymmetrical loading under cases II or III, moment distribution must be resorted to directly.

Distribution of Torsion.—The moment M_{AB} is divided in the form of torsion between the two legs l_a and l_b of the spandrel girder in the ratio of their stiffness factors. In other words, the torsion produced at end A in Fig. 3 by the member AB is divided between C and D in inverse proportion to their distances from end A. This is in accordance with Eqs. 8*b* and 8*c* and, in terms of M_{AB}, the following equations result:

$$T_a = \frac{l_b}{l_a + l_b} \times M_{AB} \qquad (13a)$$

and

$$T_b = \frac{l_a}{l_a + l_b} \times M_{AB} \qquad (13b)$$

in which the symbols have the significance shown in Fig. 3. The torsion is constant from end A to end C and from end A to end D, and of opposite sign on the two sides end A.

When two symmetrically placed floor beams frame into the spandrel girder both transmitting the same torsion, the distribution is as shown in Fig. 4. Each of the two equal outer legs of the spandrel carries all the torsion brought in by the floor beam and the middle leg has zero torsion.

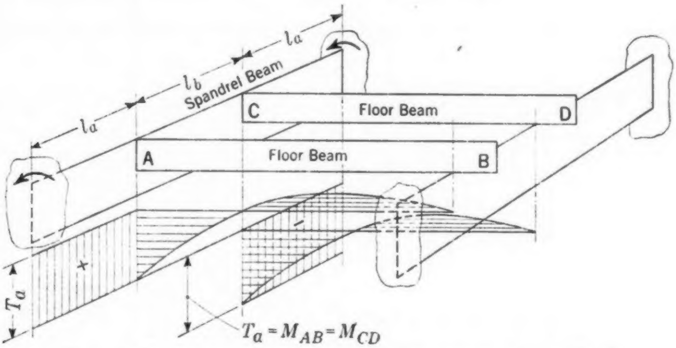

FIG. 4.—TORSION DISTRIBUTION FOR TWO SYMMETRICAL FLOOR BEAMS

Sign Convention.—A sign convention is indicated in Figs. 2, 3, and 4 which follows that of ordinary vertical shear. This convention was used to distinguish between the directions of twist on the two sides of the applied torque and to illustrate the analogy that exists between torsional and vertical shear. This sign convention applies to the spandrel beam only when the beam is treated as an isolated or separate beam. It does not apply when the spandrel is treated as a part of a space structure for the purpose of distributing moments among the end of the floor beam and the two legs of the spandrel.

Where a single floor beam frames into a spandrel, the entire length of the latter resists the moment brought in by the floor beam. Therefore, for the purpose of distributing moments in the resulting space structure, the signs of the torque on the two sides of the floor beam will be like, and will be opposite to, that at the end of the floor beam.

In the three examples below, cases I, II, and III applying Eqs. 12 are illustrated in that order. The M_{AB} for each example is first solved by the appropriate, proposed formula and then checked by the method of moment distribution (see Fig. 5, and subsequently Figs. 6 and 8). L. E. Grinter, M. ASCE, originated this scheme of applying moment distribution to space structures.[7]

ILLUSTRATIVE EXAMPLES

Example 1.—A 12-in. **WF** 106-lb spandrel girder is 10 ft long and has a 10-in. I 30-lb floor beam framing into its web 3 ft from one end. The floor beam is one of two continuous 20-ft spans rigidly connected at its ends and

[7] "Design of Reinforced Concrete in Torsion," by Paul Andersen, *Transactions*, ASCE, Vol. 103, 1938, p. 1503.

carries a uniformly applied load of 1,000 lb per ft throughout the length of both spans. Find the maximum torsional shear in the spandrel web.

This example falls under case I and, hence, Eq. 12*a* applies. From Table 1 values of R and K for the 12-in. **WF** 106-lb spandrel beam are found to be

TABLE 1.—Dimensions and Torsion Constants for Wide-Flange and Light Beams with Parallel-Sided Flanges

(Symbols Defined by Fig. 1)

Pounds per foot	Inches^4 K	Inches $m = n$
14 × 16 (R = 0.60)		
426	338.60	3.033
412	307.85	2.938
398	278.71	2.843
384	251.81	2.748
370	226.99	2.658
356	203.37	2.563
342	181.48	2.468
328	161.47	2.378
320	134.07	2.093
314	142.60	2.283
300	125.29	2.188
287	109.96	2.093
273	95.35	1.998
264	86.86	1.938
255	78.47	1.873
246	70.93	1.813
237	63.63	1.748
228	57.10	1.688
219	50.72	1.623
211	45.49	1.563
202	40.22	1.503
193	35.21	1.438
184	30.80	1.378
176	26.87	1.313
167	23.08	1.248
158	19.79	1.188
150	16.96	1.128
142	14.39	1.063

Pounds per foot	Inches^4 K	Inches $m = n$
14 × 14½ (R = 0.60)		
136	13.69	1.063
127	11.22	0.998
119	9.31	0.938
111	7.56	0.873
103	6.09	0.813
95	4.75	0.748
87	3.72	0.688
14 × 12 (R = 0.60)		
84	4.48	0.778
78	3.57	0.718
14 × 10 (R = 0.60)		
74	3.92	0.783
68	3.06	0.718
61	2.22	0.643
14 × 8 (R = 0.60)		
58	2.54	0.718
53	1.97	0.658
48	1.47	0.593
43	1.06	0.528

Pounds per foot	Inches^4 K	Inches $m = n$
12 × 12 (R = 0.60)		
190	49.96	1.736
176	39.89	1.606
161	31.20	1.486
147	23.84	1.356
133	17.95	1.236
120	13.13	1.106
106	9.23	0.986
99	7.54	0.921
92	6.09	0.856
85	4.87	0.796
79	3.90	0.736
72	2.98	0.671
65	2.21	0.606
12 × 10 (R = 0.60)		
64	2.81	0.701
58	2.14	0.641
53	1.61	0.576
12 × 8 (R = 0.60)		
50	1.82	0.641
45	1.34	0.576
40	0.97	0.516
10 × 10 (R = 0.50)		
136	26.69	1.498
124	20.37	1.368
112	15.31	1.248

Pounds per foot	Inches^4 K	Inches $m = n$
10 × 10 (*Cont'd*)		
100	11.05	1.118
89	7.88	0.998
77	5.19	0.868
72	4.23	0.808
66	3.32	0.748
60	2.53	0.683
54	1.87	0.618
49	1.39	0.558
10 × 8 (R = 0.50)		
45	1.53	0.618
41	1.15	0.558
37	0.84	0.498
33	0.59	0.433
8 × 8 (R = 0.40)		
67	5.14	0.933
58	3.37	0.808
48	1.99	0.683
40	1.13	0.558
35	0.78	0.493
33	0.65	0.463
31	0.54	0.433
8 × 6½ (R = 0.40)		
27	0.49	0.448
24	0.35	0.398

0.60 and 9.23, respectively; and $G = 0.4\,E = 11{,}600{,}000$ lb per sq in. Since only the relative values of S_T and S_F are needed for Eq. 12*a*, 29 will be used for E and 11.6 for G in Eqs. 11. The torsional stiffness factors for the two legs of the spandrel are, $S_{T3} = 1/3 \times 11.6 \times 9.23 = 35.7$ and $S_{T7} = 1/7 \times 11.6 \times 9.23 = 15.3$ (see Eq. 11*b* and Fig. 5).

Also: $S_T = 35.7 + 15.3 = 51.0$; $S_F = \dfrac{4 \times 29 \times 133.5}{20} = 773.0$; (FEM) $= 1/12 \times 1{,}030 \times 20^2 = 34{,}333$ ft-lb; $M_{AB} = \dfrac{51}{51 + 733} \times 34{,}333 = 2{,}126$ ft-lb; $T_3 = 7/10 \times 2{,}126 \times 12 = 17{,}860$ in-lb; $T_7 = 3/10 \times 2{,}126 \times 12 = 7{,}650$ in-lb; and $s_w = \dfrac{17{,}860\,(0.62 + 0.3 \times 0.6)}{9.23} = 1{,}548$ lb per sq in. The values of M_{AB}, T_3, and T_7 are found by moment distribution in Fig. 5.

Example 2.—A 16-in. **WF** 64-lb spandrel girder, 21 ft long, has a 12-in. **WF** 36-lb floor beam framing into its web at each of the third points. The floor beams are 20 ft long, are freely supported at their inner ends, and each carries an applied load of 1,750 lb per ft. It is required to review the spandrel girder for maximum torsional shear in the web.

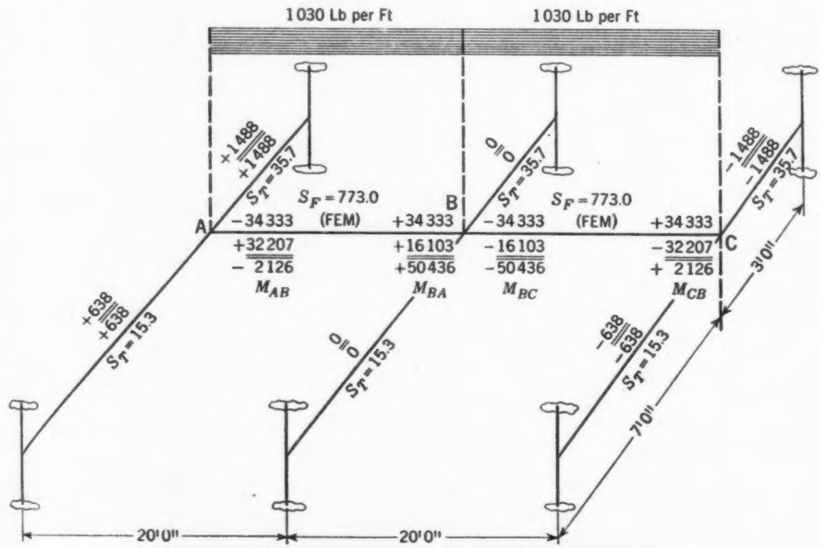

FIG. 5.—SOLUTION OF EXAMPLE 1 BY MOMENT DISTRIBUTION

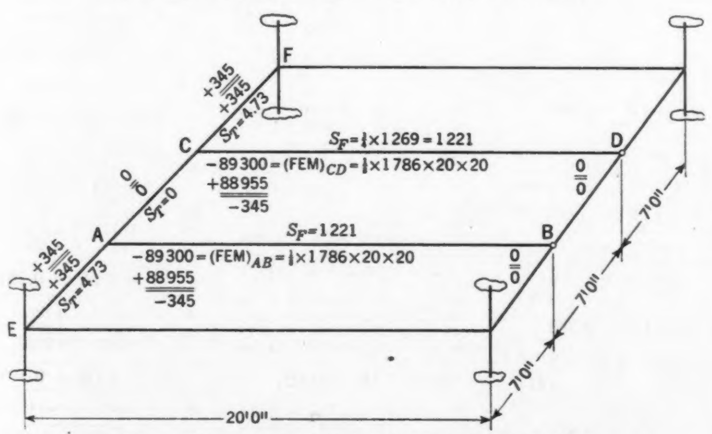

FIG. 6.—SOLUTION OF EXAMPLE 2 BY MOMENT DISTRIBUTION

As previously mentioned, only the torsional stiffness of one outer leg of the spandrel is opposed to the moment at the end of each floor beam in such a symmetrical arrangement of beams. This must be kept in mind when applying Eqs. 11*b* and 12*b*. For the 16-in. **WF** 64-lb spandrel (see Table 2), R, K, and t_w are 0.50, 2.85, and 0.443, respectively[5] (see Fig. 6).

TABLE 2.—Dimensions and Torsion Constants for Wide-Flange and Light Beams with Sloping Flanges

(Symbols Defined by Fig. 1)

Pounds per foot	Inches^4	Inches	
	K	m	n
$36 \times 16\frac{1}{2}$ (R = 0.95)			
300	68.80	1.876	1.484
280	56.43	1.766	1.374
260	44.78	1.636	1.244
250	39.68	1.576	1.184
240	35.13	1.516	1.124
230	30.94	1.456	1.064
36×12 (R = 0.75)			
194	23.69	1.402	1.118
182	19.61	1.322	1.038
170	16.03	1.242	0.958
160	13.23	1.162	0.878
150	10.77	1.082	0.798
$33 \times 15\frac{3}{4}$ (R = 0.90)			
240	39.29	1.588	1.212
220	30.33	1.463	1.087
210	26.31	1.398	1.022
200	22.78	1.338	0.962
$33 \times 11\frac{1}{2}$ (R = 0.70)			
152	13.18	1.192	0.918
141	10.36	1.097	0.823
132	8.35	1.017	0.743
125	6.88	0.942	0.668
30×15 (R = 0.85)			
210	30.69	1.494	1.136
200	26.53	1.429	1.071
190	22.85	1.364	1.006
180	19.58	1.304	0.946
172	17.01	1.244	0.886
$30 \times 10\frac{1}{2}$ (R = 0.65)			
132	10.35	1.124	0.876
124	8.52	1.054	0.806
116	6.85	0.974	0.726
108	5.35	0.884	0.636
27×14 (R = 0.80)			
177	21.57	1.357	1.023
163	16.96	1.262	0.928
154	14.42	1.202	0.868
145	12.15	1.142	0.808
27×10 (R = 0.60)			
114	7.86	1.051	0.813
106	6.32	0.981	0.743
98	5.00	0.911	0.673
91	3.89	0.831	0.593
24×14 (R = 0.70)			
160	17.82	1.303	0.967
150	14.60	1.223	0.887
140	11.88	1.148	0.812
130	9.46	1.068	0.732
24×12 (R = 0.65)			
120	8.84	1.074	0.786
110	6.92	0.999	0.711
100	5.24	0.919	0.631
24×9 (R = 0.50)			
94	5.58	0.979	0.765
87	4.46	0.914	0.700
80	3.40	0.834	0.620
74	2.66	0.769	0.555
21×13 (R = 0.65)			
142	15.27	1.251	0.939
132	12.16	1.176	0.864
122	9.71	1.101	0.789
112	7.57	1.021	0.709
21×9 (R = 0.55)			
103	8.50	1.116	0.904
96	6.86	1.041	0.829
89	5.49	0.971	0.759
82	4.32	0.901	0.689
$21 \times 8\frac{1}{4}$ (R = 0.50)			
73	3.23	0.838	0.642
68	2.62	0.783	0.587
63	2.04	0.718	0.522
59	1.67	0.673	0.477
$18 \times 11\frac{3}{4}$ (R = 0.60)			
124	12.57	1.211	0.931
114	9.93	1.131	0.851
105	7.82	1.051	0.771
96	6.01	0.971	0.691
$18 \times 8\frac{3}{4}$ (R = 0.50)			
85	5.82	1.015	0.807
77	4.42	0.935	0.727
70	3.32	0.855	0.647
64	2.56	0.790	0.582
$18 \times 7\frac{1}{2}$ (R = 0.40)			
55	1.78	0.719	0.541
50	1.34	0.659	0.481
47	1.08	0.609	0.431
$16 \times 11\frac{1}{2}$ (R = 0.60)			
114	11.01	1.173	0.897
105	8.71	1.093	0.817
96	6.75	1.013	0.737
88	5.19	0.933	0.657
$16 \times 8\frac{1}{2}$ (R = 0.50)			
78	5.08	0.976	0.774
71	3.86	0.896	0.694
64	2.85	0.816	0.614
58	2.13	0.746	0.544
16×7 (R = 0.40)			
50	1.62	0.712	0.544
45	1.19	0.647	0.479
40	0.85	0.587	0.419
36	0.59	0.512	0.344
$14 \times 6\frac{3}{4}$ (R = 0.40)			
42	1.16	0.654	0.492
38	0.86	0.594	0.432
34	0.61	0.534	0.372
30	0.41	0.464	0.302
$12 \times 6\frac{1}{2}$ (R = 0.35)			
36	0.90	0.618	0.462
32	0.64	0.558	0.402
28	0.44	0.498	0.342
25	0.30	0.433	0.277
12×4 (R = 0.30)			
22	0.301	0.443	0.405
19	0.189	0.368	0.330
16.5	0.114	0.288	0.250
14	0.072	0.243	0.205
$10 \times 5\frac{3}{4}$ (R = 0.30)			
29	0.62	0.569	0.431
26	0.46	0.519	0.381
23	0.31	0.459	0.321
21	0.23	0.409	0.271
10×4 (R = 0.30)			
19	0.239	0.413	0.375
17	0.160	0.348	0.310
15	0.106	0.288	0.250
11.5	0.050	0.223	0.185
$8 \times 5\frac{1}{4}$ (R = 0.30)			
21	0.31	0.466	0.340
19	0.22	0.416	0.290
17	0.16	0.371	0.245
8×4 (R = 0.30)			
15	0.140	0.333	0.295
13	0.089	0.273	0.235
10	0.044	0.223	0.185
$7 \times 3\frac{1}{2}$ (R = 0.30)			
12	0.112	0.364	0.282
6×6 (R = 0.30)[a]			
41	2.05	0.750	0.750
30	0.853	0.565	0.565
27	0.611	0.500	0.500
23	0.394	0.435	0.435
20	0.260	0.375	0.375
18[a]	0.177	0.343	0.285
15.5[a]	0.117	0.298	0.240
6×4 (R = 0.25)			
16	0.230	0.423	0.385
12	0.092	0.298	0.260
8.5	0.034	0.213	0.175
6×3 (R = 0.30)			
10	0.088	0.314	0.306
5×5 ($R \doteq 0.313$)			
18.9	0.348	0.503	0.330
4×4 (R = 0.25)			
13	0.156	0.364	0.326
10	0.073	0.284	0.246
7.5	0.033	0.219	0.181

[a] R = 0.25 for the 6×6 at 18-lb and the 6×6 at 15.5-lb beams.

The remaining solution then is: $S_F = \dfrac{4 \times 29 \times 280.8}{20} = 1{,}629$; $S_T = \dfrac{11.6 \times 2.85}{7} = 4.73$; (FEM) $= 1/12 \times 1{,}786 \times 20^2 = 59{,}533$ ft-lb; $M_{AB} = M_{CD} = \dfrac{2 \times 4.73}{4.73 + 1{,}629} \times 59{,}533 = 345$ ft-lb; $T_{EA} = T_{CF} = 345 \times 12 = 4{,}140$ in-lb; and $s_w = \dfrac{4{,}140\,(0.443 + 0.3 \times 0.50)}{2.85} = 861$ lb per sq in.

Example 3.—Fig. 7 represents a typical floor plan of a multi-story, steel-framed, light factory building. It has a story height of 10 ft and 8-in. brick curtain walls. All steel connections are assumed to be rigid. The floor consists of a 4-in., reinforced-concrete slab supporting a live load of 125 lb per sq ft. All beams and girders are fireproofed in concrete.

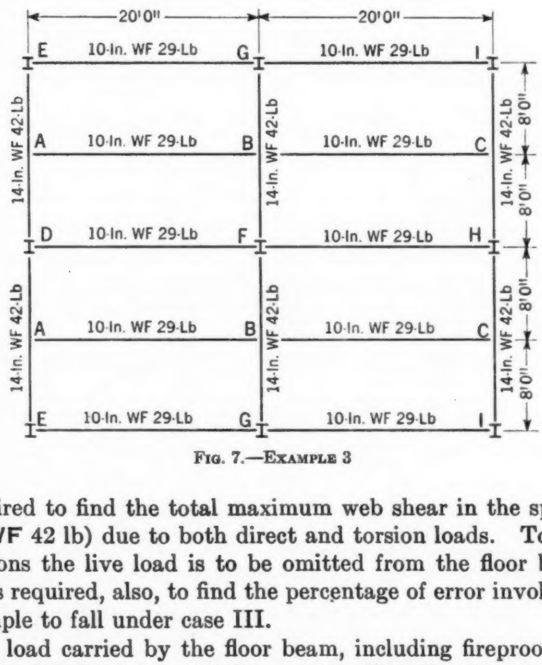

FIG. 7.—EXAMPLE 3

It is required to find the total maximum web shear in the spandrel girder DE (14-in. **WF** 42 lb) due to both direct and torsion loads. To insure maximum conditions the live load is to be omitted from the floor beam BC (see Fig. 8). It is required, also, to find the percentage of error involved in assuming this example to fall under case III.

The dead load carried by the floor beam, including fireproofing, is 520 lb per ft and the live load is 1,000 lb per ft. The K-value for the spandrel is 1.16 (see Table 2) and G is 11.6. The torsional stiffness factor for each end of the spandrel, the flexural stiffness factor for the floor beam, etc., are as follows: $S_T = \dfrac{11.6 \times 1.16}{8} = 1.68$; $S_F = \dfrac{4 \times 29 \times 157.3}{20} = 912.34$; $(\text{FEM})_{AB} = 1/12 \times 1{,}520 \times 20^2 = 50{,}667$ ft-lb; and $(\text{FEM})_{BC} = 1/12 \times 520 \times 20^2 = 17{,}333$ ft-lb.

From Fig. 8, M_{AB} is 246 ft-lb; the torque in the spandrel DE is 123 ft-lb on either side of the floor beam; and $s_w = \dfrac{123 \times 12\,(0.338 + 0.3 \times 0.4)}{1.16} = 583$ lb per sq in.

In computing the vertical shearing stress v in the web, the total shear V will be assumed evenly distributed over the web, the depth of the web being taken as the beam depth minus twice the mean flange thickness. That is,

$$v = \frac{V}{t_w (d - 2\,t_f)} \quad \text{.....................} (14)$$

It can be demonstrated that Eq. 14 gives values for the average rolled steel beam that are much closer to the maximum v given by the laborious ratio $\frac{V\,Q}{I\,b}$ than do the usual approximate formulas.

Treating the floor beam AB as a free body and taking the end moments −246 ft-lb at A and +50,998 ft-lb at B (Fig. 8) into consideration, the maxi-

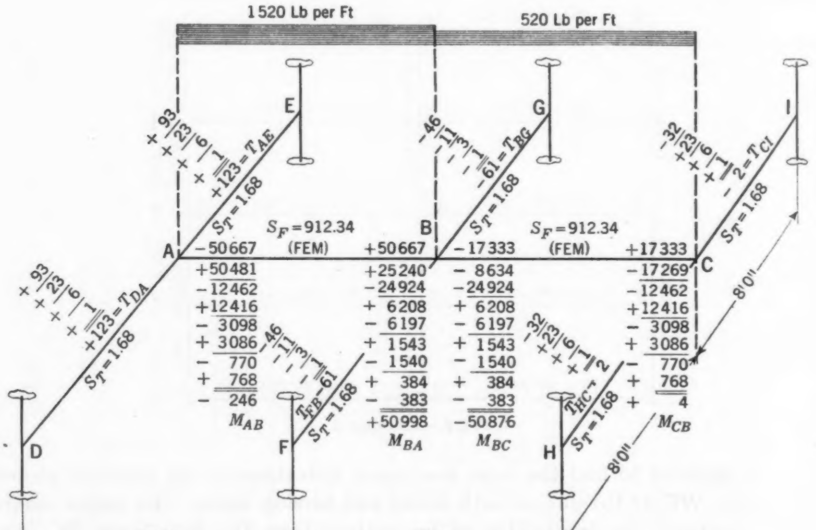

Fig. 8.—Solution of Example 3 by Moment Distribution

mum reaction that can be developed at A from the loaded floor beam AB is found to be 12,660 lb. The brick curtain wall weighs approximately 80 lb per sq ft of wall and the spandrel girder itself, including fireproofing, about 210 lb per ft; thus: $v = \frac{12{,}660 + 10 \times 16 \times 80 + 16 \times 210}{2 \times 0.338\,(14.24 - 2 \times 0.573)} = 3{,}257$ lb per sq in. The total shearing stress = 3,257 + 583 = 3,840 lb per sq in.

Short Method of Solving Example 3.—The assumption is made that loading one bay only with the live load will result in sufficient rotation of support B to give approximately 50% rigidity at that point and that, therefore, the example falls under case III. Then, $M_{AB} = \frac{3\,(1.68 + 1.68) \times 50{,}667}{2\,(1.68 + 1.68 + 912.34)} = 279$ ft-lb.

The percentage of error in M_{AB} is $\frac{279 - 246}{246} \times 100$, which is 13.4% on the conservative side. The shear is $s_w = \frac{279}{246} \times 583 = 661$ lb per sq in.; and the percentage of error in total shear is $\frac{661 - 583}{3{,}840} \times 100$, which is 2.03% on the conservative side. The percentage of error involved in neglecting the torsional shear is $\frac{583 \times 100}{3{,}840}$, which is 15.2% on the side of weakness.

Derivation of Equations for M_{AB}.—Eqs. 12*a* and 12*b* follow directly as a result of distributing the fixed-end moment of the floor beam among the end of the floor beam and the two adjacent legs of the spandrel girder. Their derivation would involve the basic principles of moment distribution. They are based on the following premises:

(1) The moment M_{AB} at the end of the floor beam is resisted entirely by the spandrel girder. In other words, the combined torsional stiffness of the two adjacent legs of the spandrel girder is opposed to the flexural stiffness of the floor beam.

(2) In order to balance the joint by the laws of moment distribution, the fixed-end moment at the end of the floor beam is distributed between it and the spandrel in the ratio of opposing stiffness factors.

(3) The sum of the torques in the two adjacent legs of the spandrel must equal the moment M_{AB} at the end of the floor beam.

(4) It can be demonstrated by moment distribution that, if the inner or end B of the floor beam, AB, is released, the resulting moment at the spandrel end is virtually twice what it would be with the inner end fixed; hence the factor 2 in Eq. 12*b*.

(5) Case III assumes 50% rigidity at the end B of the floor beam. Since this is the average of conditions assumed in cases I and II, the factor 3/2 is used in Eq. 12*c*.

Concluding Remarks

It is not good engineering practice to neglect any computable stress. Torsional shear derived from the foregoing cause may be as little as 15% of the total shear as shown in Example 3. Where the floor beam frames into the spandrel near the column, however, the shear from torsion is much more important (see Example 1). Again, it is conceivable that an intermediate stair landing might be suspended from a spandrel girder by means of hanger rods so connected as to produce torsion in the spandrel additive to that brought in by the floor beams. To neglect the torsional shear from the latter in such a case could result in an overstress.

Although this paper primarily concerns torsion in a spandrel girder, it provides, at the same time, a method for determining the moment M_{AB} at the end of the beam framing into the spandrel. It is obvious, therefore, that a method is indicated for taking the "guess" out of the end moment and the required top steel of an outside reinforced-concrete building slab (or beam) poured monolithically with its supporting spandrel girder.

Direct stress from secondary moment due to torsion was not considered. It may be found just as readily as the torsional shear by substituting the torque found by Eqs. 12 and 13, inclusive, in the appropriate equations, not included in this paper. Equations for direct stress from torsion are based on the assumption that the ends of the twisted beam are boxed in and welded to the end supports in such a manner as to prevent warping of the end sections, as well as relative movement of the two flanges at the ends. This type of connection is not used in ordinary building construction at the present time.

Other methods than moment distribution might have been used in this demonstration. The method of slope deflections, for example, would be quite adaptable. Another point about which discussion might arise is that of rigidity or extent of elasticity of beam to girder connections. The question as to whether the ordinary standard end connection is sufficiently rigid to absorb the amount of end moment involved is a legitimate one for discussion.

Some of the beam constants used in torsion equations are not given in the ordinary steel handbook. The important ones are the torsional constant K and the fillet radius R. Eqs. 6 give reasonably close values for K and may be used where tables of K-values are not available. Results that are sufficiently close for most purposes will be obtained if the following approximate values for R are used:

Depth d (in.)	Fillet radius R (in.)
20 or more	0.7
10 to 20	0.5
10 or less	0.3

APPENDIX

Notation

The following letter symbols, used in this paper, conform essentially to American Standard Letter Symbols for Mechanics, Structural Engineering and Testing Materials (ASA—Z10a—1932) prepared by a Committee of the American Standards Association, with Society representation, and approved by the Association in 1932:

b = width of section;
c = an empirical constant evaluated by substituting known values for other constants in Eq. 5;
D = diameter of a circle inscribed in a beam section (see Fig. 1);
d = depth of a section;
E = Young's modulus of elasticity;
e = eccentric distance;
(FEM) = "fixed-end moment";
G = modulus of elasticity in shear;
I = rectangular moment of inertia;

J = polar moment of inertia;
K = a torsional constant; the measure of the torsional rigidity and twisting deflections of a beam;
L = length of shaft or beam;
l = lengths along a spandrel beam; l_a and l_b are distances from a load point to either end of the beam;
M = moment; (FEM) = "fixed-end moment";
m = the greater flange thickness t_f;
n = the lesser flange thickness t_f;
P = a concentrated load;
Q = area under the moment diagram;
R = radius of fillet;
r = radius of a shaft or cylindrical beam;
S = stiffness, subscripts f and t denoting "flexural" and "torsional," respectively;
s = torsional unit shearing stress at the surface of a shaft or cylindrical beam, subscripts w and f denoting "in the web" and "in the flange," respectively;
T = torsional moment or torque;
t = thickness, subscripts w and f denoting "web" and "flange," respectively;
U = a factor depending on the $\frac{b}{n}$-ratio, subscripts L and s denoting "large end of flange" and "small end of flange," respectively;
V = total vertical shear;
v = vertical unit shear stress;
α = a factor that depends on the ratios $\frac{t_w}{m}$ and $\frac{R}{m}$;
Δ = linear displacement; and
θ = total angle of twist, with appropriate subscripts to indicate the point of application.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

ANALYSIS OF UNSYMMETRICAL BEAMS BY THE METHOD OF SEGMENTS

By Sol Lifsitz,[1] Esq.

Synopsis

A considerable number of tables and diagrams are available from which fixed-end moments, stiffness, and carry-over factors may be obtained for beams with variable moments of inertia. However, most of these tables and diagrams were prepared for symmetrical beams or for beams haunched at one end only. Since the combinations of shapes for unsymmetrical beams are limitless, it is obviously impracticable to tabulate data for but a limited range of shapes. The method proposed herein enables the designer to utilize the data available for beams haunched at one end only for the purpose of determining, quickly and accurately, the end moments, the stiffness, and the carry-over factors for unsymmetrical beams of any combination of shapes. This method should prove to be shorter than any other heretofore published for this purpose, with an accuracy equal to that obtained from the so-called "exact methods."

Nomenclature

The letter symbols used in this paper are defined where they first appear, in the text or by diagrams, and are assembled alphabetically, for convenience of reference, in the Appendix. Discussers are requested to make their nomenclature conform to these symbols. In order to avoid confusion as to signs, all terms are expressed in absolute values and the directions of end moments are indicated by arrows.

Fixed-End Moments

Divide beam AB, Fig. 1, into two segments, AO and OB, by introducing a temporary support at point O, the point of minimum thickness. Values of the fixed-end moment M_F, stiffness ratio K, and carry-over factor C, for each segment, may be obtained from available data.

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by August 1, 1946.

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Assume a vertical settlement, y_o, at point O. Compute the fixed-end moments m_F due to y_o and determine the final end moments at all the joints by the moment-distribution method.[2] Also compute F_y, the reaction at point O which is equal to, but of opposite sign to, the force acting on beam AB at point O (see Fig. 2). The downward force is simply the sum of the reactions

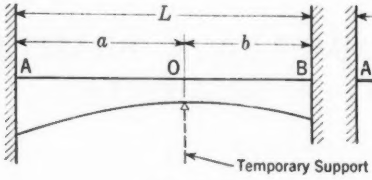

Fig. 1

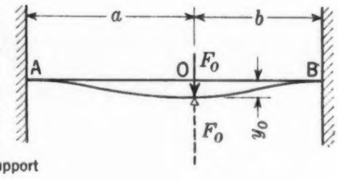

Fig. 2

at point O from the beams AO and AB resulting from the fixed-end moments in the two beams.

Fig. 3(a) shows a beam AO, fixed at point O and hinged at end A. Apply a partial moment m'_{AO} at end A, rotating joint A through the angle θ (since θ is small, $\theta = \tan\theta = \frac{y_o}{a}\Big)$. This in turn will induce a moment at the fixed joint O which is equal to m'_{AO} times the carry-over factor C_{AO}. Fig. 3(b)

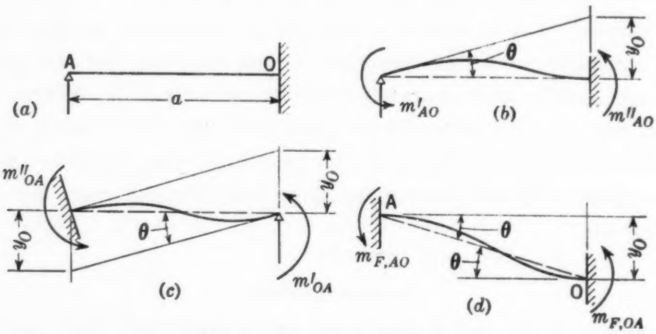

Fig. 3.—Derivation of Eqs. 1

shows the position of the beam after this rotation. The end moments at end A and end O caused by this rotation are:

$$\curvearrowleft m'_{AO} = K_{AO}\,\theta = K_{AO}\frac{y_o}{a} \qquad (1a)$$

and

$$\curvearrowleft m''_{OA} = m'_{AO}\,C_{AO} = K_{AO}\,C_{AO}\frac{y_o}{a} \qquad (1b)$$

By the principles of moment distribution:

$$K_{AO}\,C_{AO} = K_{OA}\,C_{OA} \qquad (2)$$

[2] *Transactions*, Vol. 96, 1932, p. 1.

Therefore,

$$m''_{OA} = K_{OA}\, C_{OA}\, \frac{y_o}{a} \qquad (3)$$

Lock joint A in its new position and apply a moment m'_{OA} at end O, rotating joint O through the same angle θ as shown in Fig. 3(c). The end moments from this second rotation are:

$$\curvearrowleft m'_{OA} = K_{OA}\, \theta = K_{OA}\, \frac{y_o}{a} \qquad (4a)$$

and

$$\curvearrowleft m''_{AO} = m'_{OA}\, C_{OA} = K_{OA}\, C_{OA}\, \frac{y_o}{a} = K_{AO}\, C_{AO}\, \frac{y_o}{a} \qquad (4b)$$

The final position of the beam is shown in Fig. 3(d). The final end moment at each joint is the algebraic sum of the moments at the respective joint. Therefore,

$$\curvearrowleft m_{F,AO} = \curvearrowleft m'_{AO} + \curvearrowleft m''_{AO} = K_{AO}\, \frac{y_o}{a} + K_{AO}\, C_{AO}\, \frac{y_o}{a}$$

$$= K_{AO}\, \frac{y_o}{a}\, (1 + C_{AO}) \qquad (5a)$$

$$\curvearrowleft m_{F,OA} = \curvearrowleft m'_{OA} + \curvearrowleft m''_{OA} = K_{OA}\, \frac{y_o}{a} + K_{OA}\, C_{OA}\, \frac{y_o}{a}$$

$$= K_{OA}\, \frac{y_o}{a}\, (1 + C_{OA}) \qquad (5b)$$

Corresponding formulas for the right-hand end of beam AB may be similarly derived:

$$\curvearrowright m_{F,OB} = K_{OB}\, \frac{y_o}{b}\, (1 + C_{OB}) \qquad (5c)$$

and

$$\curvearrowright m_{F,BO} = K_{BO}\, \frac{y_o}{b}\, (1 + C_{BO}) \qquad (5d)$$

Values of K and C are taken from standard tables, which are readily available. Final end moments caused by a settlement y_o are:

$$\curvearrowleft m_{AO} = m_{F,AO} + (m_{F,OB} - m_{F,OA})\, d_{OA}\, C_{OA} \qquad (6a)$$

$$\curvearrowleft m_{OA} = m_{F,OA} + (m_{F,OB} - m_{F,OA})\, d_{OA} \qquad (6b)$$

$$\curvearrowright m_{OB} = -\, m_{OA} \qquad (6c)$$

and

$$\curvearrowright m_{BO} = m_{F,BO} + (m_{F,OA} - m_{F,OB})\, d_{OB}\, C_{OB} \qquad (6d)$$

The reaction at point O necessary to restore the settlement y_o is:

$$F_y = \frac{m_{AO} + m_{OA}}{a} + \frac{m_{BO} + m_{OB}}{b} \qquad (7)$$

If a load P is applied at any point along span AO, partial end moments M' will be produced at all the joints. These are computed by distributing the

fixed-end moments at end A and at end O to all the joints. The reaction at point O is obtained from statics (see Fig. 4).

Partial end moments caused by load P acting in span AO are:

$$\curvearrowleft M'_{AO} = M_{F,AO} + M_{F,OA}\, d_{OA}\, C_{OA} \quad \text{(8a)}$$

$$\curvearrowright M'_{OA} = M_{F,OA}\, d_{OB} \quad \text{(8b)}$$

$$\curvearrowleft M'_{OB} = -\, M'_{OA} \quad \text{(8c)}$$

and

$$\curvearrowleft M'_{BO} = M_{F,OA}\, d_{OB}\, C_{OB} \quad \text{(8d)}$$

Values of M_F are taken from standard tables.

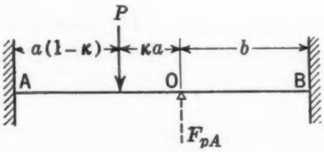

Fig. 4

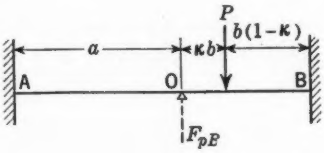

Fig. 5

The reaction at point O to resist load P in span AO is

$$F_{P,OA} = \frac{P\, a\,(1 - K) + M'_{OA} - M'_{AO}}{a} + \frac{M'_{OB} + M'_{BO}}{b} \quad \text{(9)}$$

If the temporary support is removed, point O will deflect and additional end moments will be produced at all the joints similar to those caused by the displacement y_o. Since the force required to restore point O to its original position is evidently equal to $F_{P,OA}$, it is clear that an equal but opposite force is acting downward on the beam at point O after the removal of the support. Therefore, the additional end moments created by this deflection can be obtained by direct proportion from the end moments caused by y_o. The final end moments at each joint are equal to the algebraic sum of the moments at the respective joints; or:

$$\curvearrowleft M_{AB} = \frac{F_{p,OA}}{F_y}\, m_{AO} + M'_{AO} \quad \text{(10a)}$$

and

$$\curvearrowright M_{BA} = \frac{F_{p,OA}}{F_y}\, m_{BO} - M'_{BO} \quad \text{(10b)}$$

If a load P is applied along span OB, formulas similar to Eqs. 10 are derived in the same manner (see Fig. 5). Thus, partial end moments because of load P in span OB (compare Eqs. 8) are:

$$\curvearrowright M'_{AO} = M_{F,OB}\, d_{OA}\, C_{OA} \quad \text{(11a)}$$

$$\curvearrowright M'_{OA} = M_{F,OB}\, d_{OA} \quad \text{(11b)}$$

$$\curvearrowleft M'_{OB} = -\, M'_{OA} \quad \text{(11c)}$$

and

$$\curvearrowright M'_{BO} = M_{F,BO} + M_{F,OB}\, d_{OB}\, C_{OB} \quad \text{(11d)}$$

The reaction at point O to offset load P in span OB (compare Eq. 9) is:

$$F_{p,OB} = \frac{P\,b\,(1 - K) + M'_{OB} - M'_{BO}}{b} + \frac{M'_{AO} + M'_{OA}}{a} \quad \ldots\ldots(12)$$

Final end moments corresponding to load P in span OB (compare Eqs. 10) are:

$$\curvearrowleft M_{AB} = \frac{F_{p,OB}}{F_y}\, m_{AO} - M'_{AO} \quad \ldots\ldots(13a)$$

and

$$\curvearrowright M_{BA} = \frac{F_{p,OB}}{F_y}\, m_{BO} + M'_{BO} \quad \ldots\ldots(13b)$$

Stiffness and Carry-Over Factors

To find the K-values and C-values for beam AB, the first step is to find the elastic area (A), the centroidal distance of this area ($\bar{x}$), and the moment of inertia I about the centroid of each of the two segments of the beam (see

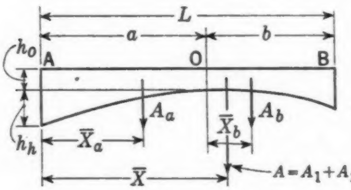

Fig. 6

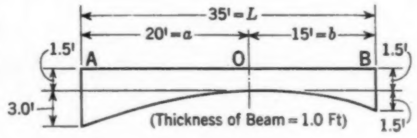

Fig. 7

Fig. 6). This is easily accomplished by the column analogy method[3] from the known values of K and C for the two beam segments; thus:

$$K_{AO} = \frac{1}{A_a} + \frac{\bar{x}^2_a}{I'_a} \quad \ldots\ldots(14a)$$

and

$$K_{OA} = \frac{1}{A_a} + \frac{(a - \bar{x}_a)^2}{I'_a} \quad \ldots\ldots(14b)$$

in which A_a is the elastic area of span AO; $\bar{x}_a$ is the centroidal distance from end A to elastic center of gravity; and I'_a is the moment of inertia of the elastic area about its center of gravity. Furthermore:

$$C_{AO} = \left[-\frac{1}{A_a} + \frac{\bar{x}_a\,(a - \bar{x}_a)}{I'_a} \right] \div \left[\frac{1}{A_a} + \frac{\bar{x}^2_a}{I'_a} \right] \quad \ldots\ldots(15a)$$

and

$$C_{OA} = \left[-\frac{1}{A_a} + \bar{x}_a\,\frac{(a - \bar{x}_a)}{I'_a} \right] \div \left[\frac{1}{A_a} + \frac{(a - \bar{x}_a)^2}{I'_a} \right] \quad \ldots\ldots(15b)$$

The signs were changed in the numerator of Eqs. 15 in order to obtain positive values for the carry-over factors. Solving Eqs. 14 and 15 simul-

[3] "The Column Analogy," by Hardy Cross, *Bulletin No. 215*, Univ. of Ill. Experiment Station, Urbana, 1930.

taneously:

$$\bar{x}_a = \frac{K_{AO}\,(1 + C_{AO})\,a}{K_{AO}\,(1 + C_{AO}) + K_{OA}\,(1 + C_{OA})} \qquad (16a)$$

$$I'_a = \frac{a\,\bar{x}_a}{K_{AO}\,(1 + C_{AO})} \qquad (16b)$$

and

$$A_a = \frac{1}{K_{AO} - \frac{\bar{x}_a}{a} K_{AO}\,(1 + C_{AO})} \qquad (16c)$$

In a similar manner the following expressions for segment OB are derived:

$$\bar{x}_b = \frac{K_{OB}\,(1 + C_{OB})\,b}{K_{OB}\,(1 + C_{OB}) + K_{BO}\,(1 + C_{BO})} \qquad (17a)$$

$$I'_b = \frac{b\,\bar{x}_b}{K_{OB}\,(1 + C_{OB})} \qquad (17b)$$

and

$$A_b = \frac{1}{K_{OB} - \frac{\bar{x}_b}{b} K_{OB}\,(1 + C_{OB})} \qquad (17c)$$

Let:

$$A = A_a + A_b \qquad (18a)$$

$$\bar{x} = \frac{A_a\,\bar{x}_a + A_b\,(a + \bar{x}_b)}{A} \qquad (18b)$$

and

$$I' = I'_a + A_a\,(\bar{x} - \bar{x}_a)^2 + I'_b + A_b\,(a + \bar{x}_b - \bar{x})^2 \qquad (18c)$$

Therefore,

$$K_{AB} = \frac{1}{A} + \frac{\bar{x}^2}{I'} \qquad (19a)$$

$$C_{AB} = \frac{-\frac{1}{A} + \frac{\bar{x}\,(L - \bar{x})}{I'}}{K_{AB}} \qquad (19b)$$

$$K_{BA} = \frac{1}{A} + \frac{(L - \bar{x})^2}{I'} \qquad (19c)$$

and

$$C_{BA} = \frac{-\frac{1}{A} + \frac{(\bar{x}\,(L - \bar{x})}{I'}}{K_{BA}} \qquad (19d)$$

Typical Example

It is required to find K_{AB}, K_{BA}, C_{AB} and C_{BA} for the unsymmetrical beam AB shown in Fig. 7; and to plot the influence lines for the fixed-end moments at end A and at end B. Data for segments AO and OA may be taken from the curves in Fig. 8 and Fig. 9. In Fig. 6, let r be a ratio of minimum beam

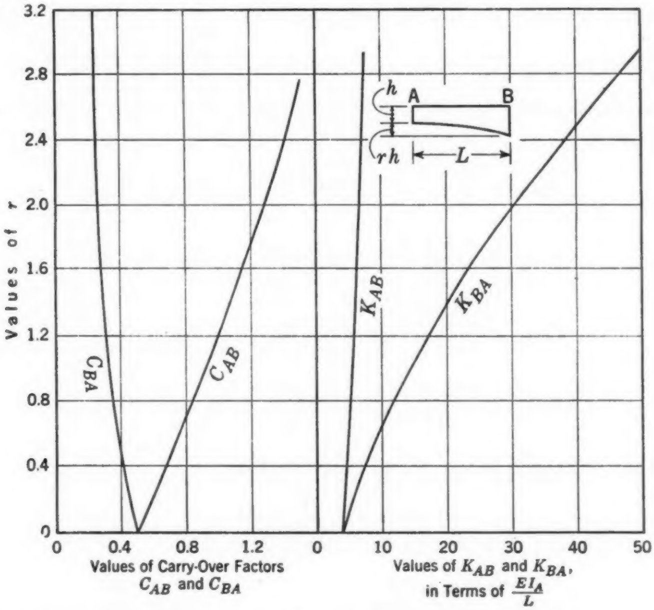

FIG. 8.—CURVES FOR THE SELECTION OF STIFFNESS RATIOS K AND CARRY-OVER FACTORS C

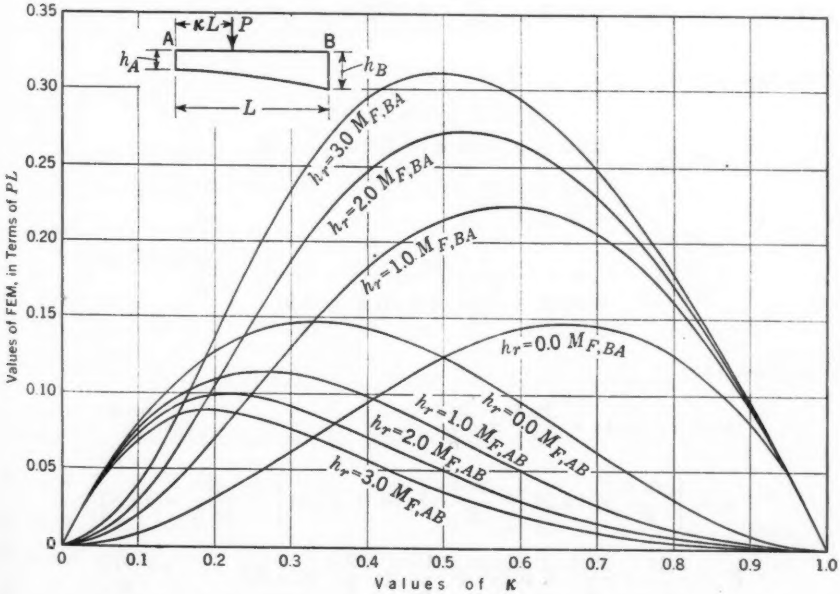

FIG. 9.—CURVES FOR SELECTION OF FIXED-END MOMENTS

depth h_o to haunch depth h_h such that $r = \frac{h_h}{h_o} - 1$. Then (see Fig. 7): $r_A = \frac{3.0}{1.5} = 2.0$; $r_B = \frac{1.5}{1.5} = 1.0$; and $I_o = 1.0 \times 1.5^3 \times \frac{1}{12} = 0.281$ ft⁴. Values for stiffness and carry-over factors at the ends of each beam segment are taken from Fig. 8:

$$K_{AO} = \frac{30.5\, E\, I_o}{20} = 0.43\, E; \qquad C_{AB} = 0.27$$

$$K_{OA} = \frac{6.5\, E\, I_o}{20} = 0.091\, E; \qquad C_{OA} = 1.27$$

$$K_{OB} = \frac{5.35\, E\, I_o}{15} = 0.10\, E; \qquad C_{OB} = 0.91$$

$$K_{BO} = \frac{14.5\, E\, I_o}{15} = 0.272\, E; \qquad C_{BO} = 0.33$$

With the foregoing values substituted in Eqs. 16:

$$\bar{x}_a = \frac{0.43\, E \times 1.27 \times 20}{0.43\, E \times 1.27 + 0.091\, E \times 2.27} = 14.51$$

$$I'_a = \frac{20 \times 14.51}{0.43\, E \times 1.27} = \frac{532}{E}$$

$$A_a = \frac{1}{0.43\, E - \frac{14.51}{20} \times 0.43\, E \times 1.27} = \frac{29.4}{E}$$

—in Eqs. 17:

$$\bar{x}_b = \frac{0.10\, E \times 1.91 \times 15}{0.10\, E \times 1.91 + 0.272\, E \times 1.33} = 5.18$$

$$I'_b = \frac{15 \times 5.18}{0.10\, E \times 1.91} = \frac{407}{E}$$

$$A_b = \frac{1}{0.10\, E - \frac{5.18}{15} \times 0.10\, E \times 1.91} = \frac{29.4}{E}$$

—in Eqs. 18:

$$A = A_a + A_b = \frac{29.4}{E} + \frac{29.4}{E} = \frac{58.8}{E}$$

$$\bar{x} = \left[\frac{29.4}{E} \times 14.51 + \frac{29.4}{E} \times (20 + 5.18)\right] \times \frac{E}{58.8} = 19.85$$

$$I' = \frac{532}{E} + \frac{29.4}{E} \times 5.34^2 + \frac{407}{E} + \frac{29.4}{E} \times 5.33^2 = \frac{2{,}612}{E}$$

—and in Eqs. 19:

$$K_{AB} = \frac{E}{58.8} + \frac{19.85^2\,E}{2{,}612} = 0.168\,E$$

$$C_{AB} = \left(\frac{-\,E}{58.8} + \frac{19.85 \times 15.15\,E}{2{,}612}\right) \times \frac{1}{0.168\,E} = 0.583$$

$$K_{BA} = \frac{E}{58.8} + \frac{15.15^2\,E}{2{,}612} = 0.105\,E$$

$$C_{BA} = \left(\frac{-\,E}{58.8} + \frac{19.85 \times 15.15\,E}{2{,}612}\right) \times \frac{1}{0.105\,E} = 0.933$$

Assuming a displacement y_o of 1,000 units at point O, Eqs. 5 yield:

$$\curvearrowleft m_{F,AO} = 0.43\,E \times \frac{1{,}000}{20}\,(1 + 0.27) = 27.3\,E$$

$$\curvearrowleft m_{F,OA} = 0.091\,E \times \frac{1{,}000}{20}\,(1 + 1.27) = 10.33\,E$$

$$\curvearrowright m_{F,OB} = 0.10\,E \times \frac{1{,}000}{15}\,(1 + 0.91) = 12.73\,E$$

$$\curvearrowright m_{F,BO} = 0.272\,E \times \frac{1{,}000}{15}\,(1 + 0.33) = 24.12\,E$$

By definition:

$$d_{OA} = \frac{0.091}{0.091 + 0.10} = 0.475$$

$$d_{OB} = 1 - d_{OA} = 0.525$$

—by Eqs. 6:

$$\curvearrowleft m_{AO} = 27.3\,E + (12.72 - 10.33)\,E \times 0.475 \times 1.27 = 28.75\,E$$

$$\curvearrowleft m_{OA} = 10.33\,E + (12.73 - 10.33)\,E \times 0.475 = 11.47\,E$$

$$\curvearrowright m_{OB} = -\,m_{OA} = 11.47\,E$$

$$\curvearrowright m_{BO} = 24.12\,E + (10.33 - 12.73)\,E \times 0.525 \times 0.91 = 22.97\,E$$

—and by Eq. 7:

$$F_o = (28.75 + 11.47)\,\frac{E}{20} + (22.97 + 11.47)\,\frac{E}{15} = 4.30\,E$$

For load P in span AO equal to 1 lb, Eqs. 8 yield:

$$\curvearrowleft M'_{AO} = M_{F,AO} + M_{F,OA} \times 0.475 \times 1.27 = M_{F,AO} + 0.603\,M_{F,OA} \quad (20a)$$

$$\curvearrowright M'_{OA} = M_{F,OA} \times 0.525 = 0.525\,M_{F,OA} \quad (20b)$$

$$\curvearrowleft M'_{OB} = -\,M'_{OA} = 0.525\,M_{F,OA} \quad (20c)$$

$$\curvearrowleft M'_{BO} = M_{F,OA} \times 0.525 \times 0.91 = 0.478\,M_{F,OA} \quad (20d)$$

Values of M_F for use in Eqs. 20 are selected from Fig. 9 and entered in Cols. 2 and 3, Table 1. The resulting values of partial end moments are entered in Cols. 4 to 7, and the remaining solution leading to influence line ordinates (see Fig. 10) appear in Cols. 14 and 15, Table 1.

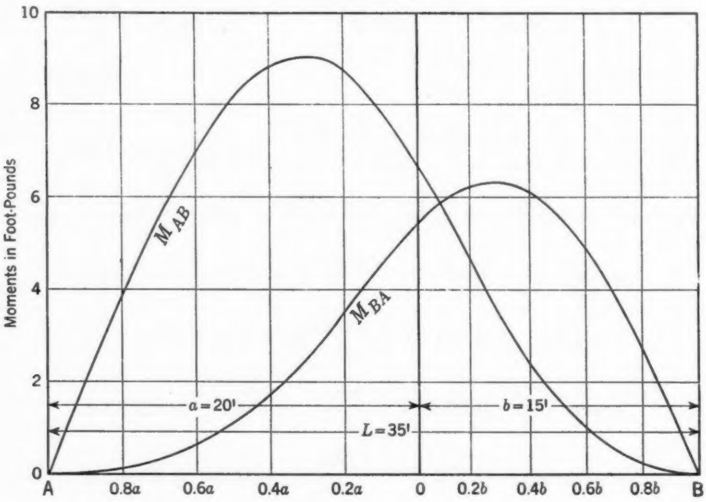

FIG. 10.—INFLUENCE LINES FOR M_{AB} AND M_{BA}

Similarly, for a load P equal to 1 lb in the span OB, Eqs. 11 yield:

$$M'_{AO} = M_{F,OB} \times 0.475 \times 1.27 \qquad = 0.603\, M_{F,OB} \tag{21a}$$

$$M'_{OA} = M_{F,OB} \times 0.475 \qquad = 0.475\, M_{F,OB} \tag{21b}$$

$$M'_{OB} = -\, M'_{OA} \qquad = 0.475\, M_{F,OB} \tag{21c}$$

and

$$M'_{BO} = M_{F,BO} + M_{F,OB} \times 0.525 \times 0.91 = M_{F,BO} + 0.478\, M_{F,OB} \tag{21d}$$

TABLE 1.—INFLUENCE OF ORDINATES FOR END MOMENTS, WITH A LOAD P OF ONE POUND IN SPAN AO

κ	FIXED-END MOMENTS (FIG. 9)		PARTIAL END MOMENTS (EQS. 8)			
	$M_{F,AO}$	$M_{F,OA}$	M'_{AO}	M'_{OA}	M'_{OB}	M'_{BO}
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0	0	0	0	0	0	0
0.2	0.103×20	0.10 ×20	3.27	1.05	1.05	0.96
0.4	0.245×20	0.072×20	5.77	0.76	0.76	0.69
0.6	0.267×20	0.032×20	5.73	0.34	0.34	0.31
0.8	0.173×20	0.009×20	3.57	0.09	0.09	0.09

TABLE 1.—(*Continued*)

κ	$a(1-\kappa) = 20(1-\kappa)$	$\frac{1}{20}$ (Col. 5 + Col. 8 − Col. 4)	$\frac{1}{15}$ (Col. 6 + Col. 7)	$F_{p,OA}$ = Col. 9 + Col. 10 or Eq. 9	$\frac{F_{p,OA}}{F_y} m_{AO} = 6.69\, F_{p,OA}$	$\frac{F_{p,OA}}{F_y} m_{BO} = 5.34\, F_{p,OA}$	Equations 10: $\curvearrowleft M_{AB}$ (Col. 12 + Col. 4)	Equations 10: $\curvearrowright M_{BA}$ (Col. 13 − Col. 7)
(1)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
0	20	1.0	0	1.0	6.69	5.34	6.69	5.34
0.2	16	0.69	0.134	0.824	5.51	4.40	8.78	3.44
0.4	12	0.35	0.096	0.446	3.05	2.38	8.82	1.69
0.6	8	0.13	0.043	0.173	1.16	0.92	6.89	0.61
0.8	4	0.026	0.012	0.038	0.25	0.20	3.82	0.11

The influence ordinates can then be computed for the case of a load P in span OB = 1 lb (Table 2), the same as demonstrated in Table 1.

TABLE 2.—Influence Ordinates for End Moments, with a Load P of One Pound in Span OB

κ	Fixed-End Moments (Fig. 9)		Partial End Moments (Eqs. 11)			
	$M_{F,OB}$	$M_{F,BO}$	M'_{AO}	M'_{OA}	M'_{OB}	M'_{BO}
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0.2	0.11 ×15	0.07 ×15	1.00	0.78	0.78	1.84
0.4	0.10 ×15	0.182×15	0.90	0.71	0.71	3.45
0.6	0.05 ×15	0.225×15	0.45	0.36	0.36	3.74
0.8	0.012×15	0.162×15	0.11	0.085	0.085	2.52

TABLE 2.—(*Continued*)

κ	$b(1-\kappa) = 15(1-\kappa)$	$\frac{1}{15}$(Col. 6 + Col. 8 − Col. 7)	$\frac{1}{20}$ (Col. 4 + Col. 5)	$F_{p,OB}$ = Col. 9 + Col. 10 or Eq. 12	$\frac{F_{p,OB}}{F_y} m_{AO} = 6.69\, F_{p,OB}$	$\frac{F_{p,OB}}{F_y} m_{BO} = 5.34\, F_{p,OB}$	Equations 13: $\curvearrowleft M_{AB}$ (Col. 12 − Col. 4)	Equations 13: $\curvearrowright M_{BA}$ (Col. 13 + Col. 7)
(1)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
0.2	12.0	0.73	0.09	0.82	5.48	4.38	4.48	6.22
0.4	9.0	0.42	0.08	0.50	3.35	2.67	2.45	6.12
0.6	6.0	0.175	0.04	0.215	1.44	1.15	0.99	4.89
0.8	3.0	0.038	0.01	0.048	0.32	0.26	0.21	2.78

Summary

The advantages claimed for this method are clearly demonstrated in the preceding example.

By readily available curves similar to those in Fig. 9, together with a few simple equations, influence-line data for fixed-end moments for any unsymmetrical beam may be quickly tabulated. The K-values and C-values for the same beam are just as easily determined by curves similar to those in Fig. 8 plus a few more simple equations.

As to the accuracy of the results obtained by this method, it is apparent that it is no less than the accuracy of the curves from which the data for the two segments are taken.

In striking contrast with the tedious computations required by the exact methods, the comparative ease with which accurate results are obtained with this method should be apparent to anyone familiar with the analysis of unsymmetrical beams.

APPENDIX

Notation

The following letter symbols, used in the paper, conform essentially to American Standard Letter Symbols for Mechanics, Structural Engineering and Testing Materials (ASA—Z10*a*—1932), prepared by a Committee of the American Standards Association, with Society representation, and approved by the Association in 1932: Subscripts *AB*, *OA*, etc. denote "in span AB," "in span OA"; or (depending on the context) "end A of Span AB," "end O of span OA," etc. Subscript *F* denotes "fixed-end"; *y* denotes "caused by settlement"; and subscript *p* denotes "caused by a concentrated load *P*."

A = elastic area of a beam $= \sum \frac{\Delta x}{E\, I_x}$;

a = left-hand segment of span L;

b = right-hand segment of span L;

C = carry-over factor;

d = distribution factor, $\frac{K}{\Sigma K}$;

E = modulus of elasticity;

F = force or reaction:

  F_y = reaction at point O of a beam necessary to restore the settlement y_o;

  F_p = reaction at point O of a beam necessary to correct the deflection caused by load P;

h = depth of a beam, with appropriate subscript to denote the section;

I = rectangular moment of inertia of the cross section of a beam $= \frac{t\, h^3}{12}$;

I_o = value of I at a point O;

I' = moment of inertia of elastic area $= \sum \frac{x^2\, \Delta x}{E^x I}$;

  I'_x = value of I' for variable segments x of a beam;

K = stiffness of a beam = moment required to rotate the simply supported end of a beam through a unit angle when the other end is fixed;

L = span length = $a + b$;

M = end moment caused by external loads P on a beam:

M_F = fixed-end moment;

M' = partial end moment;

m = end moment caused by a displacement y_o:

m_F = fixed-end moment;

m' = partial moment, right end;

m'' = partial moment, left end;

P = an externally applied concentrated load;

r = a ratio of the minimum beam depth h_o to the haunch depth h_h (Fig. 6) such that $r = \frac{h_h}{h_o} - 1$;

t = thickness, or width, of the cross section of a beam;

x = variable distance from the left end of a designated span; $\bar{x}$ = distance x to the mass center of an elastic area $= \frac{1}{A} \sum \frac{x\,\Delta x}{E\,I_x}$;

y = deflection; y_o = vertical displacement imposed on a point O;

θ = angle of rotation = $\tan \theta$;

κ = a percentage of span length (see Figs. 4 and 5) depending on the position of load P.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

FUTURE COSTS AND THEIR EFFECTS ON ENGINEERING BUDGETS

(PRESENTED BEFORE THE SANITARY ENGINEERING DIVISION OF THE ASCE ON JANUARY 17, 1946)

BY LOUIS R. HOWSON,[1] M. ASCE

SYNOPSIS

In periods of rapidly changing construction costs many essential projects fail to go ahead because budgets are insufficient to cover the cost. This is particularly applicable to the postwar period. Appropriations made in 1940, after which other than war-related construction was practically suspended, are totally inadequate to cover costs in 1946 when restrictions were lifted and work could proceed. Budgets prepared in 1946 will likewise be inadequate unless they are made after consideration of the impact of wage adjustments, increased material costs, and government policies.

This paper presents a study of the effect of other wars upon construction costs, indicates some of the factors inherent in the present situation not experienced earlier, and forecasts the level of general construction costs until 1956 in relation to costs experienced in 1940.

1. INTRODUCTION

More briefly, perhaps, the subject of this paper may be stated as "What's Ahead for the Construction Industry?" The studies here considered were prompted by considerations of developments in sanitary engineering; to a very large extent, however, the future costs of sanitary engineering works are affected by the same fundamental conditions as the cost of the construction industry in general. The effect of such increased costs on budgets and financing of what are essentially public works offers some complications which are not inherent in private work.

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by August 1, 1946.

[1] Cons. Engr., Alvord, Burdick and Howson, Chicago, Ill.

The spring of 1946 is a difficult time in which to prepare a discussion on this subject. V–E and V–J Days are so recent, and there is so much turmoil in labor relations, such delays in reconversion, and such serious maladjustments arising from the war, that accurate forecasting is impossible. As is well known it is the universal experience that there is a price "spree" after all wars. The picture in the present instance, however, is somewhat further obscured by the fact the psychological "spree" started nearly ten years before the war, and by the simultaneous adoption by the federal government of the conflicting economic policies during the war, consisting of price curbs or ceilings on commodities, combined with incentive payments to war workers.

With the war over, there is a vast reservoir of construction available. If it proceeds normally it will provide direct employment for at least 3,000,000 men, and indirectly affect many more. However, if costs rise too high, the public will refuse to pay the prevailing prices and construction will come to a dead stop. It matters little what the hourly wage of labor may be if none is employed.

Even with the critical shortage of housing, the most optimistic forecast is for the completion of 400,000 units in 1946, a figure 200,000 less than the number completed in 1940, and 300,000 less than the 10-yr average from 1921 to 1930, inclusive. The conflicting economic policies of price controls for materials and finished structures, and of higher hourly rates for labor, are found to prove difficult to reconcile.

That the volume of construction will be influenced by its cost is illustrated by a survey made for *Architectural Forum* by Crossley, Inc., which concluded "Prospects are thinking realistically and precisely about what they can afford with relation to savings and income—sizable proportions of people are inclined to postpone building or buying if the house they want is going to cost more than the price they now have in mind."

That statement is equally applicable to sanitary engineering works. It is therefore important to consider the conditions likely to affect costs and volume of sanitary works in the postwar years.

In starting consideration of this subject, reference may be made to a well-known member of the legal profession who in addressing a group of engineers stated that he always liked to be associated with engineers, for in all of his experience: "The engineers and lawyers stood back to back with the engineer facing forward." That is a brief but picturesque statement of one essential difference between the two professions. The legal profession relies almost wholly upon precedent. The engineering profession, being a creative one, necessarily makes its designs for the future. While the lessons and experience of the past are valuable guides to the engineer, construction and the period in which that construction must serve necessarily lie in the future. Every engineering undertaking is a forecast—even the appraisal of utilities already constructed and in service—for value itself is a measure of future usefulness.

Someone has defined a forecast as the art of drawing a mathematically precise line from an unwarranted assumption to a foregone conclusion. The forecast of this paper will not be of that type, for an engineer certainly ap-

preciates that any sound opinion must be built upon a foundation of information about the forces with which that opinion is concerned.

2. Necessity for Appraisal of Future Conditions

Two major elements affecting construction projects are those of time and cost. Construction projects cannot be created and put into service without incurring expenditures of money and without the lapse of time. The more extensive the project the more important becomes the factor of time.

From a long experience with municipal projects, it has become almost axiomatic that ten years ahead is the "present" and any municipal utility that is not kept that far ahead of its projected requirements is almost habitually unable to meet them as they mature. Public projects such as water supply, sewerage, and sewage treatment, therefore, are peculiarly dependent for their success upon adequate future financing as well as the development of sound engineering design and execution. In periods of rapidly rising prices scores of sound public projects fall by the wayside because the funds appropriated for their execution have proved insufficient to cover the cost of construction at the time bids are taken. This invariably results in delay, misunderstandings, criticism, and frequently, either restricting the project to a serious extent, or abandoning it for another decade or more. Adequate consideration of the factors affecting future costs and provision for them in estimates, even in such difficult times, will serve to avert some of these disappointments.

3. Construction Volume and Costs

The two important factors of construction volume and costs are closely interrelated. Construction volume is peculiarly sensitive to economic influences, the volume maxima usually occurring during periods of normal prosperity or that induced by war, and the minima normally being undertaken in times of depression when materials and labor prices are usually most favorable.

In diagrammatic form Fig. 1 shows new construction activity in the continental United States as reported in the industry report, construction and construction materials, of the Department of Commerce for the years 1929 to 1944, inclusive. Referring to the upper graph, Fig. 1, the points for 1945 and as forecasted for 1946 are from an address on "What's Ahead for 1946?" by William H. Shaw, chief of the construction statistics unit, of the construction division, Bureau of Foreign and Domestic Commerce, Washington, D. C. The figure of $15,000,000,000 for 1950 is taken from various government forecasts and construction industry goals; it will be noted that this is nearly 50% higher than the boom year of 1929, and a billion and a half dollars higher than construction expenditures during the peak war year of 1942 when new industrial plants, cantonments, and other war-related construction were under way. This is indeed an ambitious goal when expressed in terms of dollars and compared to past performance.

It is believed, however, that this upper curve by itself may be misleading, representing as it does dollars expenditure rather than the physical construction which results from that expenditure. With that thought in mind the dollars expenditure has been adjusted (lower graph of Fig. 1) to reflect the changes in

construction costs, by dividing these costs by the *Engineering News-Record* construction cost index (1913 = 100) for that year. The last four years are based upon the writer's prediction as to construction cost indexes during that period.

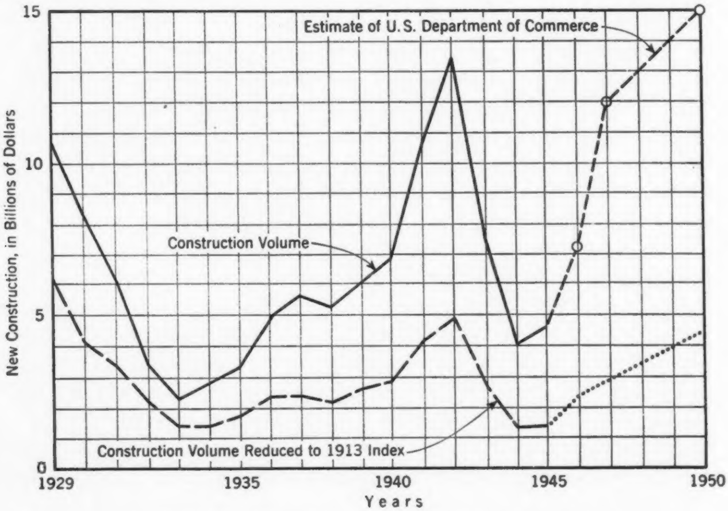

Fig. 1.—Total New Construction in United States

It will be noticed from this lower line, which reflects costs in terms of structures built rather than of dollars spent, that even the war peak year of 1942 failed by approximately 20% to reach the construction volume of 1929; and that the forecast of 1950 will be approximately 30% below the construction of 1929 and approximately 10% below the construction of 1942. In other words, although the billions of dollars which it is hoped will flow into the construction industry in 1950 will be approximately 50% greater than that of the boom year of 1929, the actual construction resulting from that huge expenditure will be about 30% less.

Public Vs. Private Construction.—During periods of normal prosperity private construction in the United States usually leads public construction in the proportion of three or four to one.

The relation between public and private construction for the seventeen years from 1929 to 1945, inclusive, is shown in Fig. 2 together with the forecast made by Mr. Shaw, for 1946. It will be noticed that in only two years, other than during the war, did expenditures for public works exceed those made by private owners; and then by only a very small amount during the worst depression years, 1932 and 1933. The forecast of the construction division of the U. S. Department of Commerce for 1946 is that 70% of the construction will be privately financed and 30% financed by the public.

In his discussion, Mr. Shaw predicted that by the end of 1946 "we can and should be building at an annual rate of better than $9,000,000,000 with

a goal of at least $12,000,000,000 in 1947." He continues with the statement, "Our forecast assumes reasonable costs—close to present levels." In appraising the possibilities therefore, it becomes pertinent to inquire as to whether costs will be maintained "close to present (1945) levels."

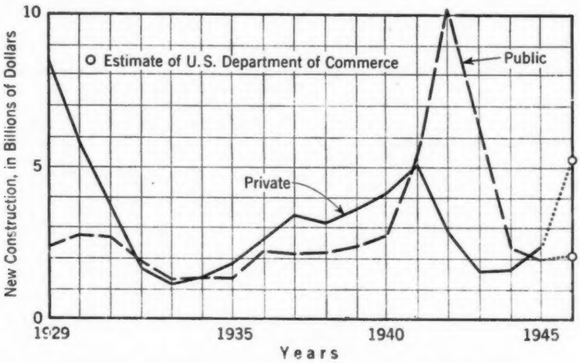

FIG. 2.—PUBLIC VS. PRIVATE EXPENDITURES FOR NEW CONSTRUCTION

4. EFFECT OF WARS ON PRICE LEVELS

Every war of lengthy duration has exerted a marked immediate effect on price levels but it has had a lasting effect as well. This is borne out by a study of recent wars and those within a period of centuries. The more nearly the present time is approached, the more accurate are the bases for the study.

The close of World War I left engineers confronted with the same type of problem that is being discussed here. At that time the late John W. Alvord, Hon. M. ASCE, was engaged on a number of public utility appraisals, one of which, the Bluefield Water Works and Improvement case, eventually found its way to the U. S. Supreme Court. The Bluefield decision was one of the two first decisions of that court holding that regulatory bodies must recognize the change in price levels resulting from the war.

A study by Mr. Alvord in 1921 at a time when construction costs of both labor and materials were at the all time peaks up to that date gave the data plotted in Fig. 3, showing the experience before, during, and after the Civil War as reflected in wages; and before, during, and up to that date as affected by World War I. Referring to the corresponding numbers on the diagram, the following notations apply:

(1) Wage statistics for period from 1840 to 1891 obtained from the Aldrich Report (Report of Committee on Finance of the U. S. Senate on Wholesale Prices, Wages and Transportation, 52d Congress, 2d Session). For period from 1892 to 1907 from the reports of the United States Bureau of Labor (*Bulletins Nos. 77 and 259*). From 1908 to 1920 from monthly *Labor Review*, March, 1921.

(2) Wage trend based on conditions preceding Civil War, ignoring effect of war.

(3) Average wage trend from 1879 to 1899; increase between curves (3) and (2) probably all due to effect of war.

(4) Wage trend based on conditions preceding World War I, ignoring effect of war.

(5) Actual wage conditions for twelve years after Civil War applied to present (1921) conditions to show probable wage curve after the war.

(6) Probable wage trend following World War I based on wage conditions following Civil War. Increase over curve (4) due to war.

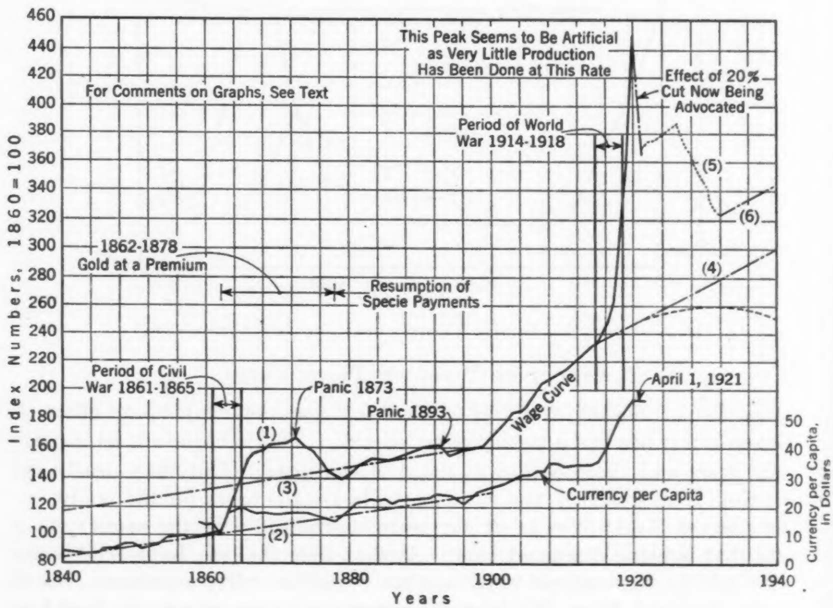

FIG. 3.—RELATIVE WAGES IN UNITED STATES, 1840–1920

The remarkable feature about Fig. 3 is that it forecasted very accurately the future wage levels, indicating a recession from 1921 to 1932 after which it predicted the upward trend would be again resumed. The low point on this diagram (1932) was approximately 145 as related to 100 for 1914, the beginning of the war. It is interesting twenty-four years after the estimate to check this forecast for the "trough" with the various construction cost indexes actually recorded for that same date. By way of comparison, some of the more important indexes for 1932 to the same base were as follows:

Engineering News-Record (E.N.-R.) Construction Cost Index	157
American Appraisal Company	155
Associated General Contractors	171

In view of its historical significance, this exhibit (Fig. 3) is included herein just as it was presented with testimony.

In Fig. 4 there are three diagrams, each starting with the beginning of an important war in which the United States was engaged. The ordinates repre-

sent indexes with the first year of the war in each case being assumed as 100, as follows:

(*A*) World War I—E.N.-R. construction cost index (1913 = 100)
(*B*) Wages before, during, and after the Civil War (1840–1891) from Aldrich Report and from 1890 to 1919 from U. S. Bureau *Bulletins Nos. 77 and 259* and monthly *Labor Review*, March, 1921.
(*C*) World War II—E.N.-R. construction cost index (adjusted to 1939 = 100)

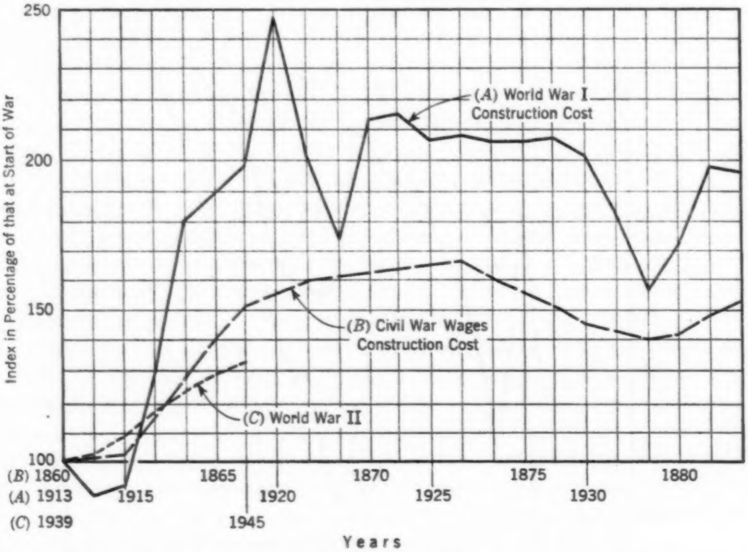

FIG. 4.—EFFECT OF WAR ON CONSTRUCTION COST AND WAGE INDEXES

The relationship and similarity during the three periods is apparent. In the Civil War and World War I aside from the extreme peak in 1920, the indexes continued to rise or maintain their level until nearly ten years after the close of the war. If it is logical to predict the future from the past, the E.N.-R. construction cost index, which as of January, 1946, was approximately 310 (1913 = 100), will rise to the 350 to 360 range by 1951 with nothing more than short time reactions in the interim.

To show the relation between wage rates and construction costs, Fig. 5 contains two diagrams, one (*A*) showing the union hourly wage rate of all building trades as reported by the U. S. Department of Labor and the other (*B*), E.N.-R. construction cost index. Just a casual glance at these diagrams, both platted with respect to the same time and index scales, will serve to show their close relationship. Construction costs have been somewhat more volatile and there would appear to be a lag in wage rates reflecting changing economic conditions, but aside from this minor difference, the relationship is remarkably close. This is as would be expected, for in the ordinary construction project

40% more or less of the total cost of project is spent for labor on the site, and a very substantial part of the remainder is expended for labor in manufacturing materials entering into the construction.

It should be noted that Fig. 5 does not reflect the result of post World War II strikes, wage increases, and cost increases. These might well raise graph B

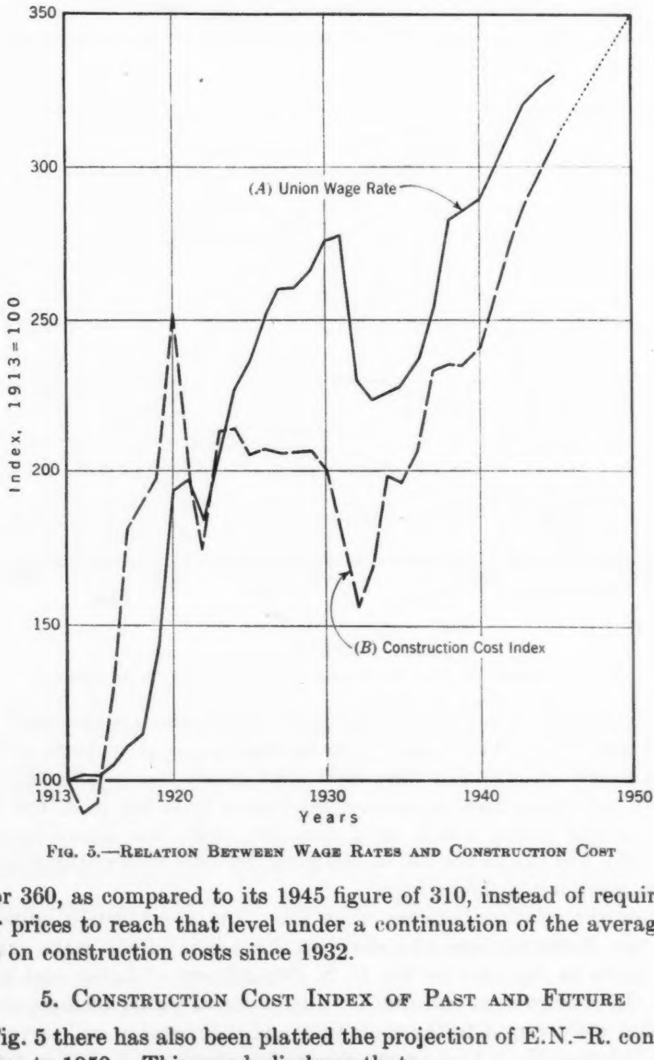

FIG. 5.—RELATION BETWEEN WAGE RATES AND CONSTRUCTION COST

to 350 or 360, as compared to its 1945 figure of 310, instead of requiring until 1950 for prices to reach that level under a continuation of the average rate of increase on construction costs since 1932.

5. CONSTRUCTION COST INDEX OF PAST AND FUTURE

In Fig. 5 there has also been platted the projection of E.N.–R. construction cost index to 1950. This graph discloses that:

(*a*) The only time since 1913 that the United States has had a substantially level construction cost index was during the period of prosperity extending through the twenties.

(b) The inclination of the construction cost index from 1932 to 1940, which is generally considered to have been a period of depression, was substantially as steep as during World War I, and World War II. This is believed to be a reflection of many factors, including general deterioration in labor output and efficiency; the growing effort of some labor union executives to discourage production incentives and piecework; the demoralizing effect of certain types of made work upon labor efficiency and morale; and various governmental efforts to keep prices up rather than permit them to react to ordinary economic laws.

It seems incongruous that, during the period covered by one of the worst depression periods ever experienced in this country, construction costs should rise more than 50%. With the prices of such basic commodities as corn and hogs depressed to 20% of normal and with 10,000,000 men out of work, construction costs increased at the rate of 10% per year. Contractors went out of business and sanitary engineering construction work was so revolutionized that for a period of nearly ten years not a foot of sewer was laid by contract in the City of Chicago (Ill.) The Work Projects Administration (W.P.A.) with its demoralizing effect on labor efficiency had effectively supplanted contract work. Public works were largely subsidized by federal funds and minimum wages were instituted as a condition for the subsidy. During the nine years from 1932 to 1940, inclusive, public expenditures for construction averaged 75% as much as private. This compares to a normal of from 30% to 40%.

In 1945 Harry E. Jordan, Affiliate ASCE, executive secretary of the American Water Works Association, in an effort to secure a cross section of opinion as to why the $2,000,000,000 municipal public works program was not progressing more rapidly to the blueprint and construction stages, sent a questionnaire to 150 consulting engineers, to engineers of the various state departments of health, and to others interested in water works and sewerage construction. The replies were prompt and comprehensive. In general, the reasons for the delay fell into three rather well-defined channels:

1. Rising costs of construction.
2. Uncertainty as to whether there would be federal participation in public works after the war.
3. Congestion in designing engineers' offices.

The first reason, that of rising costs, is fully discussed herein. The second reason, resulting from the uncertainty as to federal policies, can only be removed by prompt action in Washington and of that there seems to be little likelihood. The third reason, congestion in designing engineers' offices, is gradually ironing itself out.

Uncertainties Affect Contract Costs.—It was almost the universal experience during 1945 that such sanitary engineering work as went out to contractors for bids attracted few bidders and their prices were materially higher relative to prewar amounts than would be indicated by the increase in the E.N.-R. construction cost index. This reflects the uncertainty and fear inherent in the present picture. Uncertainty costs money. Contractors have their four fears: (1) Rising material prices; (2) inability to get materials when needed; (3) rising labor wage rates; and (4) decreased efficiency of labor.

Availability and efficiency of labor will probably improve somewhat after the unemployment benefits to war industry workers are stopped. It is believed that the only important recession from present high construction contract costs will be brought about by stabilizing wages and thus stabilizing prices of materials but that the general construction cost tendency thereafter will be upward for the next few years.

6. Conclusions

From the foregoing discussion it is believed certain general conclusions may be drawn:

1. It is apparent that hesitancy and uncertainty, particularly in the labor picture, both as to cost and efficiency, justify the reluctance of contractors to bid on work of any magnitude requiring any considerable length of time, unless a very liberal "contingency" factor is included. Washington's "no policy" labor attitude is believed to be largely responsible.

2. It is believed that clarifying the labor picture will greatly improve the possibility of securing fair bids on construction work; that wage stabilization even at a higher level will affect contractors' bids less than the previous uncertainty.

3. Engineers should be realistic. Postwar construction costs are bound to be materially higher than prewar. In most lines of general construction the increase above 1940 will probably approximate 50% by 1950, and in certain classes of construction such as pipe lines and sewers involving a large proportion of common labor, the increase will be materially higher.

4. Aside from the removal of the contingent item by the clarification of the labor situation, there is nothing in the picture of prospects for 1946–1956 to indicate that construction costs will be materially lowered. There will probably be a continued rise, possibly accelerated by the further rise in labor wage scales.

5. Rising construction costs particularly to an extent in excess of 50% above 1940 will materially retard construction and many public works will be deferred or abandoned.

6. One of the important deterrents toward normal procedure in public works construction is the failure of Washington to declare itself with respect to federal aid. Aside from a few of the very large cities most American cities do not want federal aid for their projects. However, many public works will be deferred awaiting a definite position relative to federal aid for public projects. Public officials do not want to be in the position of having their city proceed on its own when had it waited a few months it could have had a government subsidy which would either have reduced that city's cost or enabled it to do more work.

7. So far as sanitary engineering works are concerned, about $2,000,000,000 worth of water works and sewerage construction was in the development stage as of January 1, 1946, of which something less than 10% was in the blueprint stage. Cash was in hand for much of the water works improvements. Sewerage, however, was not so fortunate; much of this work was bound to be delayed as it was projected on the basis of 1940 costs which had already been exceeded by from 30% to 60%.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

FACTORS CONTROLLING THE LOCATION OF VARIOUS TYPES OF INDUSTRY

By Charles P. Wood,[1] Esq.

Synopsis

Any discussion of factors controlling the location of industry must take into account new and potent influences, resulting from World War II, which have disrupted a normal way of life and have changed the scope of future industrial activities. Revolutionary improvements in methods and equipment will be the constructive results of research and manufacturing experience growing out of the war effort. New reservoirs of labor and new potential manufacturing areas have been created by the development of war industries remote from established industrial centers. A reorganized distribution system, already geared up to export tremendous quantities of material from all parts of the United States through both Pacific and Atlantic ports, will facilitate decentralized and diversified industrial operations.

New opportunities, improved processes, and other causes of progress are familiar experiences in the American business and manufacturing field. The period from 1914 to 1940 seemed at one time to include a cycle of war and depression and recovery. The greatest war in history promptly followed, and resources and markets, once thought to be assured for the future, are no longer to be taken for granted. Factors that affect the location of industry are subject to changeable influences.

The Prospect of Changed Conditions

The rearrangement of the industrial pattern in the United States has been predicted from time to time without sufficient allowance for practical considerations. In defining the elements that have most to do with the location of industries, care must be taken to avoid categorical statements. The younger generation of engineers should study the history and economics of industries,

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **August 1, 1946.**

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as well as the technical theories, to guard against the mistakes of their predecessors.

Too many industrial plans are still dependent on the same way of life that was in effect during the past generation. The most important changes affect markets, transportation, labor, and management. Changes in processes and equipment are radical and far reaching, but they will be much easier to make than the changes that depend on providing a new kind of industrial environment for workers and a new system of distribution. Perhaps the greatest difference between the old and the new is the conception of time.

The soldiers, sailors, Wacs, and Waves of World War II will form the dominant element of management and of the working force as well as of the domestic market. Some of them made the trip from India, the Philippines, Japan, and China within two days—the same time their parents were brought up to think would be required to go from New York, N. Y., to Houston, Tex., or from Chicago, Ill., to San Francisco, Calif. This conception of time is affected by other innovations developed during World War II. Short cuts made possible by electronics and radio, seeing in the dark with radar, automatic devices for doing what formerly depended on the human senses and labor, and airborne freight—all these have combined to change much that once made up the requirements which affect the location of industries.

Fundamental Requirements

In spite of all these innovations and improvements, many old difficulties remain unsolved and the factories of the future will require power and labor, transportation facilities, and a supply of materials. Discrimination should be made between the type of enterprise that is tied to rigid requirements and the one that is free to follow the lines of least resistance.

The basic industries, which process raw materials in large bulk, are still confined to locations where transportation of raw materials to the plant and transportation of finished products to the market can be accomplished with greatest economy. Transportation in the broad sense includes power transmission lines, pipe lines for oil and gas, and facilities for assembling labor, as well as freight carriers. The conversion from raw material to the final product generally proceeds in separate steps which may occur at several different locations, depending on the nature of the product and the market to be served.

The Steel Industry

The steel industry offers the best example of integration, because the manufacturing process can be traced from definite sources of raw material to a variety of products without departing from the main channels of commerce and industry. Ore and coking coal for the manufacture of pig iron can be assembled most economically at certain blast furnaces. The larger plants, of course, carry the processes through to the steel ingot and on to the rails, plates, sheets, structural shapes, rods, and wire. Steel plants away from the blast furnaces take advantage of local supplies of scrap and fuel in combination with transportation facilities. Plants must be located to insure economical distribution

of the product, as well as delivery of pig iron from the blast furnace to the open hearth or electric furnace whenever the local supply of scrap is not sufficient.

After steel has been made into rods, wire, plates, sheets, and structural shapes, it is sent to the fabricating plant, whose location depends entirely on the local market or on local resources that make manufacturing most economical. For example, Detroit, Mich., consumes steel for automobiles; New Britain, Conn., steel for hardware, and Newport News, Va., steel for ships. The location of these factories and shipyards is governed by factors different from those that govern the location of the steel plant. In some places conditions are favorable for blast furnaces and steel mills (Fig. 1), as well as for miscellaneous manufacturers of steel products, including ships—for example, Sparrows Point, Md., where blast furnaces on the coast are supplied with imported ore and domestic fuel. The Great Lakes area, using domestic ore and fuel, offers similar advantages, except that the size of ships built on the lakes is limited by the width and depth of the Welland Canal. Blast furnaces and steel plants operating on the Pacific Coast are handicapped by high transportation cost of fuel, and to a lesser extent that of ore. The scale of operations in peacetime, however, will depend on local markets and the location of bases for freight rates, as well as on transportation costs and competition.

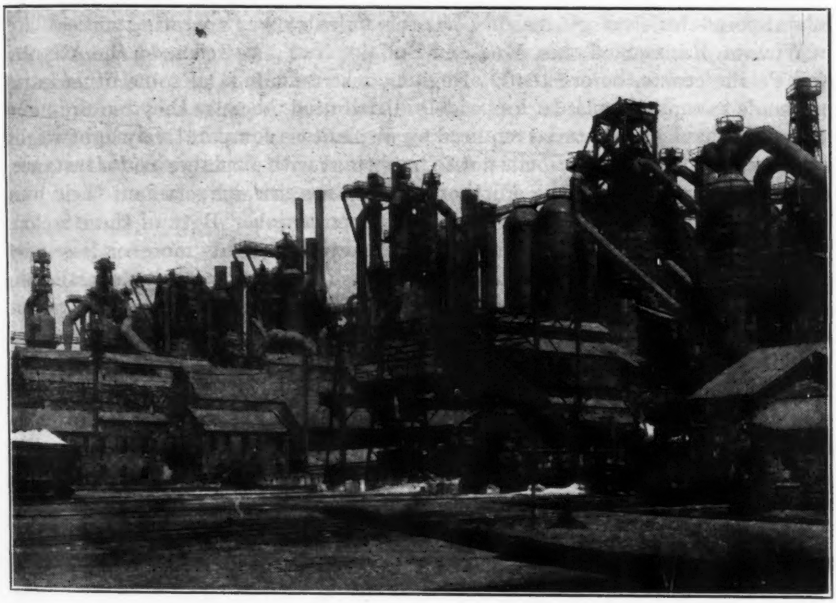

FIG. 1.—"IN SOME PLACES CONDITIONS ARE FAVORABLE FOR BLAST FURNACES"
(General View of Blast Furnaces, Carnegie-Illinois Steel Company)

PROCESSING INDUSTRIES

Portland cement, lime, sugar, flour, and coffee roasting offer examples of processes in which transportation again is a controlling influence. Milling-in-

transit rates have served to prevent concentration of such processing industries around the comparatively few inland sources of raw materials or at seaports. The effect of these rates is to make it possible to manufacture the product somewhere between the source of material and the market—the exact location depending on the loss in bulk during the various processes and the best centers of distribution.

The Airplane Industry

The airplane industry grew up on the Pacific Coast very much as the automobile industry did in Detroit—that is, through the ability and enterprise of local individuals. The war accelerated the manufacture of airplanes at inland points for strategic reasons, but this industry was operating successfully in Wichita, Kans., St. Louis, Mo., and Buffalo, N. Y., as well as on the Atlantic and Pacific coasts, before 1940. Engines, instruments, and some other parts are made in separate plants, less widely distributed, because they require more skilled mechanics. Material required for airplanes is comparatively lightweight and the cost of transportation is not so important with airplanes as, for instance, with the heavy machinery. Furthermore, planes are delivered on their own power independently of existing surface transportation. Both of these factors tend to make the location of airplane manufacturing plants more or less independent of the sources of material and of surface transportation. Nevertheless, progressive management is probably the main reason why airplane manufacturers have broken away from traditional limitations.

The airplane industry developed its own force of skilled labor more effectively than did other industries which followed one another into districts where a supply of skilled labor was known to exist. Most of the workers in airplane factories are young men and women who have become sufficiently skilled without going through a long apprenticeship. Even airplane engine factories, where a nucleus of highly skilled mechanics is required, were operated in wartime at places where few comparable precision operations had been performed before. The industry deserves the distinction of leading the way to decentralization of manufacturing by a combination of boldness and ability which is characteristic of the younger element in management.

Miscellaneous Small Industries

Miscellaneous machinery manufacturing may follow the example of the airplane industry to a certain extent, and may show a preference for locations that serve the market although retaining other essential advantages, such as a supply of skilled mechanics. Again, improved transportation is a factor and skilled labor will find it easy to follow opportunities for employment and better living conditions. This discussion should include also industries which are composed of small units, comprising a large number of individual plants with a large combined volume of products and a large number of employees. In a plan for the industrial development of a district, these smaller, diversified industries become more important than the industries characterized by large and impressive individual plants. Manufacturers of garments and textile specialties—shoes, radio, electrical appliances, styled products, and novelties—

are dependent primarily on labor, markets, and distribution. The cost of transporting raw materials and the cost of power (which are vital to the success of some industries) are not among the most important factors in these industries. Studies to determine the location of some of the comparatively small plants are more difficult than corresponding studies in which a few large items clearly determine the most important considerations. In these smaller plants, involving smaller expenditures for each of a larger number of items, every slight advantage has to be considered, because success or failure may depend on any one of a number of slight advantages over competitors.

The Textile Industry

The textile industry came to New England from England because management and mechanical skill were available and the climate was supposed to be suitable. However, the effect of climate has been eliminated by humidifying and air conditioning systems, which are standard equipment for textile mills. In the Southeast the textile industry began to take advantage of lower operating costs, and the Southeast became competitive with New England as New England had become competitive with Europe. The cost of labor is important in the textile industry which is one of the industries that can train skilled operators within a comparatively short time. Therefore, locations where labor is plentiful, with a consequently low cost, are sought. The Southeast offered a supply of good textile labor at comparatively low wages, with other conditions favorable to the industry; and so cotton mills migrated from New England to the South, causing southern mills to constitute the greater part of the cotton textile industry. When these mills were built, wages were extremely low in the South and the regulation of working hours was comparatively lax or liberal. The costs of power, construction, food, and shelter also were low. Since the adoption of uniform national laws governing wages and working hours, southern mills no longer enjoy a competitive position based primarily on low wages and long hours. The better mills do not depend on these differentials, because good management and productive labor can overcome them. Uniform wage-and-hour laws have put the textile industry on a different basis with respect to location. In fact, markets and distribution facilities should be more important factors in the location of textile mills.

The textile industry is comparable to the steel industry with respect to the relationship between manufacturing yarns and fabrics in the basic plants and to the distribution of these materials throughout a wide variety of manufacturing operations, whose locations are controlled by influences different from those which affect the basic plants. For example, cotton mills convert raw cotton into yarn. Some mills make a variety of yarn to be shipped to plants that weave specialities in comparatively small quantities. The larger mills, making large quantities of standardized products, combine weaving with spinning in a completely balanced plant. After the fabrics have been woven, they are finished in bleaching, dyeing, and printing plants which are generally separate from the mill. The products of these finishing plants are distributed to the garment makers and miscellaneous fabricators and consumers. The manufacture of rayon yarn is a chemical operation with locational requirements

different from the requirements of weaving plants. Rayon yarn is made in comparatively large plants and rayon weaving is widely distributed. Both rayon and wool manufacturing fall into the same general divisions as cotton—namely, yarn manufacturing, weaving, finishing, and the conversion of finished fabrics into marketable products.

Textile yarn manufacturing requires power and labor at competitive rates, together with large manufacturing sites in localities where conditions contribute to reasonable living expenses. Weaving plants have the same requirements for power and labor at competitive rates, especially in the larger units, but their requirements are more flexible. Textile finishing plants require large supplies of soft water and good transportation facilities from the mills and to the market—thus they are found in, or accessible to, distribution centers. The final products are made in plants of various sizes generally situated where they serve the market to best advantage. This accounts for the concentration of needle trades in the large cities, especially New York, which is the largest market for textile products.

Shoes, Radio, and Appliances

The shoe industry, which began in New England and later expanded westward, enjoys more freedom than the textile industry in the choice of plant location, because it consists in the main of smaller units. It is another major manufacturing industry in which plant location will be influenced, by markets and distribution facilities, in the future more than it has been in the past.

Manufactures of radios, household appliances, and numerous other attachments for the convenience and amusement of the individual provide a variety of employment for labor released by war industries and by the Army and Navy, irrespective of manufacturing locations. This condition should have a stimulating effect on the creation of new enterprises and on employment in areas not formerly considered active in the industrial field.

Statistical References

The "Census of Manufactures"[2] includes statistics from which industries can be classified according to their requirements for labor, fuel, and power. More than four hundred classes of industry are listed including the sum spent for wages, fuel, and power; the number of male and female employees; and the value added by manufacture in each class. Another convenient reference is "Industrial Location and National Resources."[3]

Statistics from the census can be used to compute the number of wage earners, the wages paid, and the cost of fuel and power per unit of value added by manufacture. A study of these results will show which industries employ the largest number of workers, pay the highest wages, or spend the most for fuel and power on the basis of value added by manufacture. It may be assumed that these industries will seek locations where labor, fuel, and power are available on the most economical basis. Intelligent inspection of the

[2] "Census of Manufactures," U. S. Dept. of Commerce, Washington, D. C.

[3] "Industrial Location and National Resources," National Resources Planning Board, U. S. Government Printing Office, Washington, D. C., 1943.

industries so segregated should eliminate cases for which statistical results are not confirmed by practical requirements. Further study of these computations will show certain coincidences. Two or three of these elements—for example, wages paid, number of employees, and cost of fuel and power—may combine to indicate preferences for locations having corresponding advantages.

When the number of wage earners alone is relatively high and the wages paid do not exceed the average, it may be assumed that the industry is not especially dependent on highly paid or skilled labor. Where the reverse is true and the wages paid are comparatively high although the number of employees is comparatively low, the industry probably requires skilled or high-priced labor.

The cost of fuel and the cost of electric power should be combined for general classifications because the census does not go into sufficient detail to discriminate between industries that generate all or part of their power and others in which the processes or local conditions make purchased power more economical. Generally, power is cheap where fuel is cheap, and so the combined expenditure for power and fuel is an index to the advantages of locations where these two items are available at a low cost.

The census also shows the number of male and female employees in various classes of industry. Selecting those industries in which more than 50% of the employees are female and then comparing the list with others showing comparatively large numbers of employees, or comparatively large sums paid for wages, will lead to interesting conclusions on the characteristics of employment in existing industries. These conclusions, considered in connection with the local population and labor supply, should serve as a basis for determining which industries are most desirable in a community.

Application of Fundamental Requirements

Familiar examples can be cited to confirm conclusions from these census studies. Products of mass production and products in which the value of material is relatively high do not appear in the list showing large numbers of employees and high costs of fuel and power per unit of value added by manufacture. Automobiles, airplanes, precision machinery, and jewelry are industries comparatively free from other restrictions on location but dependent on a supply of skilled labor. Woodworking, canning, food products, soap, and cheap novelties are industries requiring a comparatively high number of employees. Boatbuilding, railway car building, styled clothing, foundries, leather, and musical instruments are industries with comparatively high wages. Some of the industries in which both the number of employees and the wages paid are comparatively high are buttons, textiles, clothing, drugs, and medicines.

A comparatively high cost of fuel and power is indicated for the manufacture of electrochemical and electrometallurgical products, ceramic products, steel products, refined sugar, chemicals, vegetable oils, textile finishing, paper, and ice. The last item is included to illustrate an exception. The location of an ice plant is determined more by accessibility to the market, on account of expensive handling and loss during shipment, than by the availability of cheap power.

Ceramic and leather products are the most important industries showing a combination of heavy pay rolls and high cost of fuel and power per unit of value added by manufacture. The combination of a comparatively large number of wage earners with a heavy pay roll and high cost of fuel and power is characteristic of the textile industry. The foregoing examples can be multiplied by increasing the scope of the study, but these few will suffice to show the wide variety of industrial activities that share the same fundamental requirements with respect to power, fuel, and labor.

Transportation is a primary requirement for nearly every industry, whether involving raw materials, finished products, communications, or means for taking employees back and forth to work. Transportation costs include, besides the usual freight and passenger charges, the costs of time consumed and of damage to perishable or fragile products, such as ice. The time element, during the nonproductive transportation period, affects the amount of working capital tied up by work in process and by materials and products in transit. Transportation facilities, however, are not limited to railroads, automobiles, airplanes, or other locomotive appliances. They include parking space, landing fields, shelter for passengers, and shipping and receiving facilities. Limited accommodations for parking and poor transportation equipment between passenger terminals and factories are marks of obsolescence that apply to many of the older plants.

Influence of Climate

The effect of climate on the location of industry is often the converse of what might be expected. The most highly developed industrial districts are in the more severe climates, when it might be expected that temperate climates would have attracted population and provided economical conditions under which industry would prosper more than in places subject to severe weather. The conclusion is that life in severe climates makes people more adaptable to industrial work than they would be if they lived in mild or warm climates, which are particularly favorable to agriculture. There are many exceptions to this rule and the influence of fuel and mineral deposits is recognized. Still, the volume and variety of manufacturing in Scandinavia, Northern Europe, New England, and the Great Lakes district, not approached in other more temperate or warmer climates, proves that climatic conditions are not controlling factors in the location of manufacturing plants.

Opportunities for Planning

There is an important relation between city planning and factors controlling the location of various types of industry. By city planning, desirable local conditions can be created where they might not exist otherwise.

The activities leading to the adoption of a city plan, as well as the execution of the plan itself, should have a wholesome effect on industrial prospects. Major considerations, such as power, freight rates, availability of materials and accessibility of markets, wage scales, supply of labor, and even housing and living conditions, may balance about equally when comparing several desirable

places. The advantages of one over the other will then depend upon plant sites, local laws and taxes, local utilities, amusements, and local transportation —to mention only a few of many possible items that are susceptible of intelligent treatment in a comprehensive plan.

Industrial districts can be laid out with railroad sidings and highways arranged so that factory sites of various sizes can be accommodated and so that the individual plants are relieved from building long spur tracks and paved roads, water, gas, and sewerage lines, and electric power lines to connect with the nearest existing system.

It is a mistake to select industrial locations where the cost of land might restrict the size of the site. Room is needed for parking space, future expansion, protection against hazardous storage, and for other reasons which may develop long after the site has been chosen. For example, helicopter landings, as yet unusual, promise to become typical features of future industrial sites. The principal reasons for the use of sites that are too small are high unit cost of land and prohibitive cost of providing transportation and utilities at other sites. City planning can eliminate these major difficulties by the proper location and development of industrial districts, as well as by proper zoning of land conveniently situated for industrial and commercial use.

The disposal of waste from industrial processes has been neglected in the past but such neglect is no longer permissible. There are few sites, however isolated, where it is legitimate to allow an industry to dump offensive waste. It is much better to face the problem wherever new plants are built than to evade it in the hope that somebody else will do something about it later. The pollution of streams and bathing beaches and the contamination of water supplies have created serious problems wherever provision has not been made for the treatment or disposal of objectionable waste. At the same time, regulations which are too stringent cause the industries concerned to favor locations where the cost of compliance will be more reasonable. Therefore, it is no service to a community to formulate restrictions or to enforce penalties which discourage desirable development. Neither is it any service to an industry to allow it to become a nuisance and then to prosecute it for an offense which could have been avoided.

Fair taxation should be inherent in any plan for the most desirable as well as the most profitable use of land. Industries may seek locations outside city limits primarily to avoid city taxes but, at the same time, they get more land without paying too much for the plant site. It should pay the city to let industry have plenty of room. The city's income is not necessarily reduced, in such cases, by an amount corresponding with what the industries could have been taxed if within the city limits. The city gets its income indirectly, because wages paid by the industry are spent in the city and support the expansion of retail business and services, the construction of homes, and the consumption of goods sold within the city. Sound economics would prescribe locations outside the city limits for industries needing large sites or room for expansion, so the city eventually would get the benefit of increased expenditures for wages and materials.

There is a large and important class of industries which seek locations within the city limits. Such industries show a preference for the congested districts accessible to concentrated labor reservoirs, markets, and services, and so they provide tenants for loft buildings, warehouses, and buildings specially constructed to accommodate small manufacturing tenants. Some of these industries continue to be comparatively small, although others expand later and seek larger sites and more commodious facilities in outlying areas.

The so-called incubator building, or manufacturing loft building, is a favorite subject for discussion in this connection. Experience has proved that such buildings are important assets to communities when they are built correctly and in the right location. Failures may be caused by spending too much on these buildings as well as by poor building design and construction and by improper location. It should be assumed that the tenant wants only the essentials for manufacturing convenience and comfort of employees. These loft buildings should be designed with liberal floor loads, with flexible systems of power, water, steam, and gas connections, and with provision for ample entrances and exits and for compliance with insurance regulations. The wiring, piping, and ventilating systems should have a capacity sufficient for air conditioning equipment, which can be installed later as needed. Railroad sidings and shipping platforms for cars and trucks, protected from the weather and served by conveniently placed elevators, are other important requirements. City planning, making it possible to find desirable sites for these buildings and access to them without blocking traffic in adjacent streets, will create inducements for important new industries. More than that: It will improve the appearance and the convenience of the city and increase the value of central property.

Summary

The location of industrial plants is subject to new influences resulting from World War II.

Certain fundamental requirements, including the supply of materials, labor, power, fuel, and transportation, retain their relative importance.

A trend toward decentralization is apparent in the effort to relieve congestion, to serve new and expanding markets, and to utilize new reservoirs of labor and improved transportation facilities.

Local and regional planning should provide for improved and economical sites for both large and small industries.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

THE PLANNING OF AERIAL PHOTOGRAPHIC PROJECTS

By F. J. Sette,[1] M. ASCE

Synopsis

Although there is a progressively developing literature in the field of photogrammetry, until the studies herein described were initiated, there was practically no information in regard to planning and estimating the cost of aerial photographic projects. This paper discusses the establishment of a photographic day, zones of similar weather characteristics, and a method of scheduling projects. The reliability of the representative photographic day is discussed. A method of estimating the cost of aerial photography is also presented.

Introduction

The extensive use of aerial vertical photography by the U. S. Department of Agriculture for crop control, soil conservation, flood control, forestry studies, and other important purposes led to an examination of that program with a view to reducing its cost if at all possible. More than 2,900,000 sq miles of the United States have been photographed—approximately 700,000 sq miles of which has been photographed twice because of cultural and other changes on the ground. For certain purposes it may be necessary to rephotograph an area more than once. A decision to rephotograph would be based upon the extent of cultural changes since the date of the original photography, the requirement of greater pictorial detail, or the need for more precise, or larger scale photographs. Aerial photography offers the engineer a useful and economical tool in the planning and execution of engineering projects; in fact, it is a necessary tool which will be increasingly useful in the future.

Unit costs developed in this paper are based upon 1938 prices; upon a photographic scale of 1 to 20,000 (1,667 ft per in.); and the use of a camera with a focal length of $8\frac{1}{4}$ inches. For this scale and camera, the flying altitude is 13,750 ft above the ground.

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **August 1, 1946.**

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SCHEDULING OPERATIONS

Photographic Day.—The number of days in a month that an aerial photographer could reasonably expect to utilize for photographic purposes had to be determined. The Soil Conservation Service (SCS) of the Department of Agriculture had obtained from the United States Weather Bureau a 10-yr record of the number of days for some fifty stations throughout the United States when the sky was cloudless or less than 10% overcast—or, assuming the perfectly clear sky to be unity or 1, when clouds covered 0.1 of its area or less as an average of three observations taken at 7 a.m., noon, and 7 p.m. Flying

TABLE 1.—METHOD OF GROUPING[a] STATIONS OF SIMILAR WEATHER CHARACTERISTICS

Weather station	Days	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
(a) REGION 2 (SEE MAP IN TABLE 2)													
New York, N. Y.	4.5	91	102**	109**	96	102**	71	71	87	120**	158†	100**	91
Atlantic City, N. J.	6.3	97	98	97	89	103**	71	84	89	113**	157†	116**	94
Baltimore, Md.	5.4	93	104**	100**	94	94	70	81	78	118**	168†	111**	100**
Washington, D. C.	5.3	100**	100**	100**	93	93	72	72	83	117**	164†	111**	100**
Norfolk, Va.	6.2	95	106**	100**	105**	106**	74	65	68	92	157†	139**	102**
Cape Henry, Va.	6.4	91	86	91	105**	106**	77	81	89	106**	152†	123**	87
Hatteras, N. C.	7.0	86	87	104**	107**	110**	86	86	79	91	137**	126**	93
Philadelphia, Pa.	5.4	104**	107**	100**	96	104**	67	72	74	117**	156†	107**	89
Harrisburg, Pa.	4.6	89	89	109**	98	100**	63	81	91	143**	163†	91	76
Elkins, W. Va.	2.6	81	65	115**	119**	142**	64	58	64	108**	154†	127**	77
Richmond, Va.	6.5	97	100**	95	106**	100**	66	65	69	108**	165†	118**	100**
Lynchburg, Va.	4.0	88	90	102**	107**	95	60	90	83	112**	163†	105**	95
(b) REGION 3 (SEE MAP IN TABLE 2)													
Raleigh, N. C.	6.5	105**	102**	102**	109**	91	52*	46*	55*	98	182†	155†	102**
Chattanooga, Tenn.	5.3	94	104**	110**	85	91	58	45*	55*	96	196†	160†	106**
Charlotte, N. C.	6.5	103**	103**	111**	97	94	43*	37*	46*	98	186†	160†	120**
Columbia, S. C.	6.0	95	108**	105**	107**	92	57	38*	52*	93	176†	167†	118**
Atlanta, Ga.	6.0	105**	110**	120**	97	88	60	32*	40*	88	175†	163†	117**
Birmingham, Ala.	6.4	103**	108**	117**	100**	102**	64	44*	47*	81	177†	153†	97
Montgomery, Ala.	6.7	100**	98	112**	104**	91	66	42*	51*	102**	184†	157†	102**
Mobile, Ala.	5.7	103**	98	102**	105**	104**	65	30*	42*	102**	186†	156†	103**
Meridian, Miss.	6.4	95	91	108**	95	87	86	48*	62	103**	186†	145**	97
Vicksburg, Miss.	6.4	100**	91	105**	100**	89	84	50*	56	102**	181†	148**	95
Wilmington, N. C.	7.0	109**	109**	116**	109**	101**	63	43*	51*	71	153†	160†	124**
Charleston, S. C.	6.3	111**	118**	124**	119**	102**	56	35*	43*	70	151†	167†	116**
Savannah, Ga.	6.6	106**	112**	129**	126**	106**	64	36*	41*	68	139**	160†	114**
Augusta, Ga.	6.7	109**	109**	107**	109**	103**	61	37*	54*	91	167†	154†	110**
Macon, Ga.	7.2	103**	108**	117**	100**	102**	64	44*	47*	81	177†	153†	111**
Pensacola, Fla.	6.3	100**	102**	110**	110**	108**	67	29*	40*	92	173†	159†	111**
New Orleans, La.	5.2	112**	102**	112**	117**	100**	63	19*	23*	92	190†	170†	105**

[a] First column shows the average number of days per month that the sky is 0.1 or less overcast. The numerals in the other columns show the percentage of the average which may be expected for the stated month. Groups suggesting similar weather patterns may be identified as follows—*, 55% or less; **, 100 to 149%; and †, 150% to 199%—of an average month.

reports from the field seemed to indicate that the number of days having a sky covered with 0.1 or less of clouds could be used as a very close index of the number of photographic days that might be expected. Accordingly, the number of stations was extended to include all stations for which records were available. The years of record varied from 26 to 37, the largest number of stations having a 37-yr record.

When this decision was made, it was also appreciated that an aerial photographer could utilize other days when he often found himself unable to photo-

graph on so-called "clear days" because of adverse local conditions such as haze, fires, heavy winds, or mechanical and other difficulties. Therefore, the designation of a photographic day as that day in which clouds covered 0.1 or less of the total sky served largely as an index.

Regions of Similar Weather Characteristics.—For each weather station, the average number of days per month for the year ($\frac{1}{12}$ average number of days in a year) was divided into the actual average for each month which, multiplied by 100, gave the percentage that each month was of the average for the year. In other words, the values in Table 1 are percentages of the average days in the first column of the table. For example, Table 1 shows that, in an average year, New York, N. Y., has 4.5 days per month when the sky is 0.1 or less overcast—clear days. The average March weather is clearer than that, being 109% of 4.5 days or an average of 4.9 days. Stations with similar characteristics are indicated by appropriate superscripts. The number of stations (144) covering the entire United States made it difficult to ascertain the boundaries between regions. To do so, probability curves were plotted for each station. The curves for some of the stations were of the normal fre-

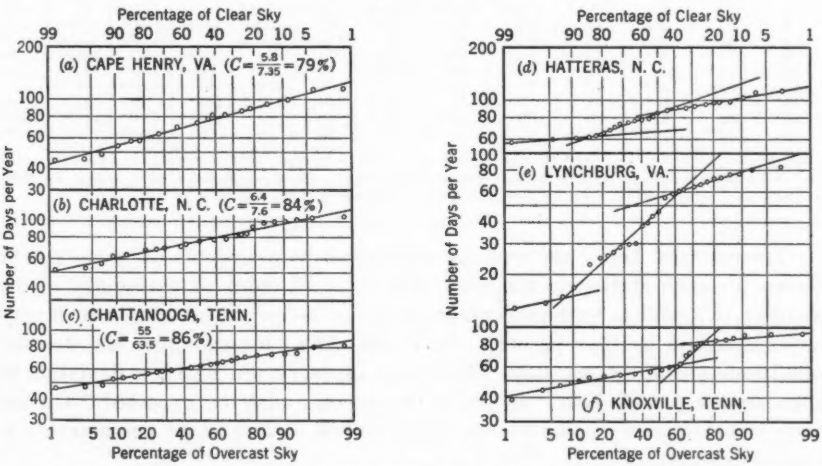

FIG. 1.—DETERMINATION OF BOUNDARIES OF AREAS OF SIMILAR WEATHER CHARACTERISTICS BY PROBABILITY CURVES
(C = COEFFICIENT OF VARIATION)

quency distribution type whereas those for other stations were not. It was assumed, therefore, that, for the former stations, the weather pattern was consistent whereas the latter stations were subject to two or more weather patterns. Such irregular areas, therefore, were assumed to be boundary stations. Figs. 1(*a*), 1(*b*), and 1(*c*) illustrate the consistent type of station; and Figs. 1(*d*), 1(*e*), and 1(*f*), the boundary type of station.[2] There seems to be no definite boundary line, but rather a transition zone between the regions. Since there was no need to be precise about region boundaries, these

[2] "Method of Estimating Time Required on Aerial Photographic Projects," by F. J. Sette, *Photogrammetric Engineering*, Vol. IV, No. 1, p. 42.

were drawn through those stations which showed the greatest variation in normal regional distribution.

As a result of these studies, Table 2 and Fig. 2 were prepared. Table 2 offers a quick method of scheduling projects. For example, regions 3 and 4 show very poor conditions for the months of June through September when two or three days per month is the average expectancy of good photographic weather, whereas in November, six to eleven days may be expected. Consequently, considerations of cost should dictate the scheduling of photographic projects in these areas from October through April. Equipment released from these areas could be used in regions 5 and 6 through October.

TABLE 2.—DATA REQUIRED FOR PREPARING A SCHEDULE OF OPERATIONS IN AERIAL PHOTOGRAPHY[a]

Region	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1	100**	112**	125**	109**	95	66	69	94	115**	141**	85	89
2	94	95	102**	101**	105**	70	76	80	112**	158†	115**	92
3	104**	104**	112**	106**	97	62	36*	47*	90	175†	159†	108**
4	121**	149**	162†	139**	97	52*	28*	28*	44*	97	152†	131**
5	27*	38*	86	121**	155†	137**	150†	138**	154†	127**	43*	24*
6	79	73	100**	110**	124**	113**	122**	117**	131**	122**	60	49*
7	91	84	85	86	83	80	93	100**	125**	162†	118**	93
8	94	86	82	82	81	87	118**	115**	124**	135**	100**	96
9	116**	91	84	64	55*	79	74	68	142**	160†	135**	132**
10	115**	88	94	92	98	111**	42*	47*	104**	148**	134**	127**
11	50*	52*	65	75	86	127**	159†	160†	151†	132**	87	56
12	38*	53*	58	77	78	102**	226††	212††	155†	110**	51	40*
13	90	77	93	99	103**	113**	85	82	116**	129**	110**	103**
14	123**	100**	108**	94	63	52*	56	76	99	128**	159†	142**

[a] Groups that suggest similar weather patterns may be identified as follows: *, 55% or less; **, 100% to 149%; †, 150% to 199%; and ††, 200% or more of average month.

The numeral below the average number of photographic days per month shown for each station in the map, Fig. 2, is an index of variability of the weather (C), and is that percentage above or below the average which may be expected in a 10-yr period. At Yuma, Ariz., for example, the weather conditions are fairly constant. One year in every ten years, a deviation of approximately 14% below or above the average may be expected. On the basis of below average conditions, 18.4 ($= 0.86 \times 21.4$) days per month is a normal expectation.

Validity of Index.—Although some preliminary observations had indicated that the number of days 0.1 overcast or less may be equivalent to the number of photographic days that could be expected, it was decided to test this assumption on the large-scale operations of the 1938 aerial photographic program of the Agricultural Adjustment Administration (AAA). Weekly weather reports of daily cloud conditions over the project areas were obtained by arrangement with the Weather Bureau. Four such areas are defined in Table 3(*a*). The study that follows is based largely on weekly reports of operation received from the contractors and from the Weather Bureau observers nearest the project locations. The reports from the crews were really estimates and were considered as such. Also, Weather Bureau stations were often located at a distance from the area being photographed; it was possible for perfect weather


Eastport 3.1 48%
Portland 6.7 27%
Boston 5.7 34%
Nantucket 1.6, 88%
Block I. 5.7, 46%
Northfield 2.5 47%
New Haven 6.6, 35%
New York 4.5, 23%
Philadelphia 5.4, 23%
Atlantic City 6.3, 40%
Cape Henry 6.4, 32%
Norfolk 6.2, 32%
Hatteras 7.0, 27%
Canton 4.9 37% 46%
Albany 5.1 46%
Oswego 4.4
Binghamton 3.1 41%
Scranton 5.2 37%
Harrisburg 4.6, 25%
Baltimore 5.4 23%
Washington 5.3, 26%
Richmond 6.5 28%
Rochester 3.9 28%
Buffalo 2.7, 54%
Erie 3.6 35%
Pittsburgh 3.7, 50%
Parkersburg 3.9 38%
Elkins 2.6, 49%
Lynchburg 4.0, 66%
Raleigh 6.5 21%
Wilmington 7.0 21%
Charleston 6.3, 30%
Charlotte 6.5 23%
Columbia 6.0 32%
Savannah 6.6 25%
Jacksonville 5.5 25%
Miami 2.9 54%
Tampa 4.9 44%
Key West 5.4, 41%
Augusta 6.7 28%
Macon 7.2 27%
Atlanta 6.0 22%
Knoxville 5.4 26%
Chattanooga 5.3 19%
Cleveland 5.9 23%
Sandusky 4.9, 38%
Columbus 4.9, 25%
Toledo 5.2, 21%
Detroit 4.1, 32%
Lansing 4.4, 27%
Alpena 3.2 34%
Sault Ste. Marie 3.8 23%
Marquette 2.7 52%
Escanaba 4.8 33%
Louisville 6.4, 24%
Cincinnati 5.2, 25%
Indianapolis 4.8, 29%
Evansville 5.5 32%
Nashville 6.2 27%
Birmingham 6.4 36%
Montgomery 6.7 26%
Pensacola 6.3 30%
Mobile 5.7 27%
Meridian 6.4 29%
Memphis 8.0 27%
Chicago 5.5, 25%
Milwaukee 6.0 26%
Springfield 6.4, 18%
St. Louis 7.2 24%
Cairo 5.7, 24%
Davenport 6.3, 28%
Dubuque 5.4 32%
La Crosse 5.0 29%
Duluth 5.3 36%
Minneapolis 5.1 33%
Charles City 5.4 55%
Keokuk 7.6, 27%
Columbia 7.2, 25%
Kansas City 8.0 24%
Springfield 10.1 23%
Des Moines 5.4 32%
Sioux City 6.3 36%
Omaha 6.7 26%
Topeka 8.7 29%
Fort Smith 7.4 22%
Little Rock 7.3 24%
Vicksburg 6.4 26%
New Orleans 5.2 31%
Shreveport 8.4 22%
Galveston 6.6 33%
Palestine 6.5 30%
Corpus Christi 5.8 39%
Fort Worth 7.4 35%
San Antonio 6.1 27%
Moorhead 6.7 47%
Huron 6.3 29%
Devils Lake 5.4, 37%
Bismarck 5.9 26%
Valentine 6.0, 31%
Lincoln 6.4, 28%
Concordia 6.4 24%
Wichita 7.6, 35%
Oklahoma City 7.9 24%
Abilene 8.2 27%
Del Rio 7.7 30%
North Platte 8.1 38%
Dodge City 9.1, 52%
Amarillo 6.5 70%
Rapid City 6.6 29%
Williston 5.4 58%
Miles City 7.4 25%
Sheridan 6.6 34%
Cheyenne 5.3 27%
Denver 6.1 19%
Pueblo 7.0 31%
Roswell 10.2 31%
El Paso 12.5 19%
Santa Fe 6.8 37%
Lander 5.4 33%
Havre 4.9 44%
Helena 4.3 35%
Yellowstone Park 4.7 30%
Grand Junction 8.3 18%
Kalispell 4.7 26%
Pocatello 6.6 27%
Salt Lake City 9.0 31%
Modena 10.3 34%
Phoenix 14.6 20%
Yuma 21.4 14%
Spokane 4.6 33%
Walla Walla 6.1, 50%
Boise 8.0 19%
Baker City 6.8 25%
Winnemucca 10.6 28%
Seattle 3.6 31%
Portland 4.5 24%
Roseburg 6.5 26%
Reno 12.3 16%
Eureka 4.3 34%
Sacramento 15.2 24%
San Francisco 8.7 34%
Fresno 15.4 16%
Los Angeles 10.0 17%
San Diego 8.5 31%
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FIG. 2

to exist in the area being photographed while the sky was more or less overcast at the station where the observations were made.

Tables 3(*b*) and 4 should be studied together. The former shows the number of flights made and the available photographic days as reported by

TABLE 3.—STUDY OF FLIGHT CONDITIONS FOR AERIAL PHOTOGRAPHY, BY PROJECT AREAS

No.	Description	East Central[a]	North Central	Southern	Western	Totals
	(*a*) DEFINITION OF PROJECT AREAS					
1	States included	New England New York Pennsylvania New Jersey Delaware Kentucky Maryland North Carolina Tennessee Virginia West Virginia	Illinois Indiana Iowa Michigan Minnesota Missouri Nebraska Ohio Wisconsin South Dakota	Alabama Arkansas Florida Georgia Louisiana Oklahoma South Carolina Texas Mississippi	Washington Oregon California Nevada Idaho Utah Oregon New Mexico Colorado Wyoming Montana North Dakota Kansas	....
	(*b*) NUMBER OF AVAILABLE PHOTOGRAPHIC DAYS (SKY, 0.1 OR LESS OVERCAST)					
2	Number of flights....	659	1,080	621	517	2,877
3	Clear days[b]........	762	776	787	648	2,973
	(*c*) TIME REQUIRED, TOTAL AND AVERAGE					
	Hours:					
4	En route..........	1,325	2,173	1,159	1,222	5,879
5	Photography......	1,383	2,636	1,408	1,185	6,612
6	Total...........	2,708	4,809	2,567	2,407	12,491
	Hours per Flight:					
7	En route..........	2.01	2.02	1.87	2.36	2.04
8	Photography......	2.10	2.44	2.27	2.29	2.31
9	Total...........	4.11	4.46	4.14	4.65	4.35
	(*d*) AREA COVERAGE					
10	Square miles[c].......	137,236	261,784	121,997	113,441	634,458
	Square Miles per:					
11	Flight...........	204	242	197	219	220
12	Photographic hour..	99.1	99.4	86.7	95.7	96.1
	(*e*) NET SQUARE MILES PER EXPOSURE					
13	Number of exposures.	53,396	111,302	84,460	53,392	....
	Square Miles:					
14	Total photographs..	85,939	149,857	85,138	68,400	....
15	Per exposure.......	1.61	1.35	1.01	1.28	....

[a] Includes the northeastern United States. [b] Reported days, 0.1 or less overcast. [c] Reported total coverage.

the Weather Bureau; and the latter discloses that only 54.8% of the total flights, accounting for 61.4% of the total photography, occurred on days of 0.1 or less overcast.

Two factors are important in appraising the value of the reported cloudiness: (1) The method of averaging 7 a.m., noon, and 7 p.m. observations, as used by the Weather Bureau; and (2) the location of the Weather Bureau observer, who might have been at considerable distance from the area being photographed. Thus, it is perfectly possible that, at the 7 a.m. observation, it was raining or that the sky was completely overcast, clear by 10 a.m., and 0.2 overcast at 7 p.m. Such a day would be reported as 0.4 overcast although it would be excellent photographic weather. Similarly, the cloud conditions over the area as a whole may have been 0.1 or less overcast and 0.4 overcast where the observer was located. Moreover, with the present availability of highly sensitive photographic film (which is several times "faster" than that produced a few years ago) complete cloudlessness is no longer an absolute requirement; on the contrary, sufficiently high cirrus clouds sometimes have a beneficial effect on the quality of the photographs by softening the light and reducing the intensity of the shadows.

TABLE 4.—NUMBER OF FLIGHTS AND THE AREA COVERED, AS RELATED TO CLOUD CONDITIONS

(As Reported by the Weather Bureau from the Project Area)

Proportion of sky overcast[a]	Number of flights	Square miles covered[b]	PERCENTAGE OF TOTAL:	
			Flights	Coverage
0.1[c]	1,576	389,334	54.8	61.4
0.2	383	82,011	13.4	12.9
0.3	309	58,953	10.7	9.3
0.4	221	44,212	7.7	7.0
0.5	143	22,994	5.0	3.6
0.6	117	16,062	4.1	2.5
0.7	59	9,555	2.0	1.5
0.8	25	2,991	0.8	0.5
0.9	27	4,462	0.9	0.7
1.0	17	3,884	0.6	0.6
Total	2,877	634,458	....	....

[a] Average cloudiness. [b] Reported total coverage. [c] 0.1 of sky, or less, overcast.

A study of the maximum 2-day performance of each contractor, totaling one hundred and one flights, showed an average coverage of 630 sq miles and 5.06 photographic hours per day as compared with 220 sq miles and 2.31 hours for all flights. These hundred and one flights (or 3.5% of all flights) accounted for 63,501 sq miles, or better than 10% of the total coverage. The reported cloud conditions of the 101 days when these records were made showed 84 days, that were 0.1 or less overcast and 11 days that were 0.2 overcast. All factors considered, it seems evident that for all practical purposes the index (namely, the day of 0.1 clouds or less) has substantial validity as a photographic day.

Effect of Sunlight.—Thus far no mention has been made of the effect of season, or of varying hours of sunlight, upon the time available for photography. In general, photography in the northern latitudes ceases about November 15 (when snow may be expected or when the photographic time is too short to continue operations economically), and is rarely resumed before April 1.

In the southern states, between latitudes 27° to 36° north, where only a comparatively few days of snow coverage is normal, the length of photographic daylight is of primary importance. There is no period in these latitudes between the dates of January 21 and November 22 when there is not at least three full hours of sunlight (10:30 a.m. to 1:30 p.m.) and when the sun is not

less than 30° above the horizon—a requirement that is generally specified in order to avoid objectionable shadows. There is a 2-hr period (11:00 a.m. to 1:00 p.m.) between December 4 and January 7 when the sun has a vertical angle of not less than 30° above the horizon. At latitude 30° north, the sun is 30° or more above the horizon for at least 3 hours during the winter months, which is more than the average time of photography (2.27 hours) in Table 3(*c*) for the southern region.

Little is known of the effect of winter sun on the quality of photography; but it is well known that when the sun is low the rays are obscured by considerable haze and other poor atmospheric conditions. The shadows cast by the winter sun are long, and they obscure photographic detail. The best photographic time is the midday period of a clear day following precipitation, and the best season is as nearly June 21 (the summer solstice) as clearing of snow from the ground and the leafing of the trees will permit.

Tables of solar altitudes and Weather Bureau reports should be consulted when planning aerial photographic projects extending into the winter months.

Photographic Coverage

General Considerations.—To prepare a program and to estimate an aerial photographic project properly, it is necessary to know what the plane and the crew can do. The area that a crew can photograph depends on many factors, some of which are: Speed and power of plane, experience and ability of crew, scale of photograph, elevation of plane required to obtain the necessary scale, topography of area (mountainous or flat) and its mean elevation, whether or not the area is marked by the outline of public land surveys of the United States General Land Office, availability of good flight maps, favorable weather conditions, and distances from flying base to area to be photographed.

In general, the faster a plane can climb to its position and the speedier it is, the more time can be spent on photography, and more photographs can thus be obtained in a given length of time. Better advantage can also be taken of favorable weather conditions.

The most important single factor is the ability of the crew. Good pilots for aerial vertical photography are not numerous because of the severe combination of qualities required of them. Only a small percentage of trained pilots seem to have the patience and mapping intuition of line and direction necessary for success in aerial photography. The photographer and pilot must work as one unit, which adds to the difficulty of obtaining many successful combinations. Consequently, the area that can be photographed in a given time depends very largely on the skill of the crew because it involves not only the original photography but also the additional reflying necessary to fill in gaps or to replace unacceptable work.

Areas that are sectionized by the Public Land Survey System are more easily photographed from the air because the pilot can use section lines to guide him. In general, the states that are not sectionized, in addition to the thirteen original states, are West Virginia, Kentucky, Tennessee, Vermont, Maine, and parts of Texas and Louisiana.

Good flight maps enable the members of the crew to plot the flight lines, and to select landmarks as guides in the air. If good maps are available, they should be supplied to the contractors. When rephotography of an area is to be undertaken, probably the best flight maps would be the "photo-index" sheets obtained from previous projects. These should be supplied to the crew.

Flying Time.—Of first consideration is the flying time of a single photographic expedition. Table 3(*c*) indicates that, on the average, it takes 2.04 hours for a plane to leave the ground, fly into photographic position, and return to the ground after completing its mission. For brevity, this interval may be called "en route time." Approximately 2.31 hours are required for photography, making a total of 4.35 hours for the complete flight.

It is rather interesting to note that, in the western division, the en route time required was appreciably above the average (2.36 hours), probably because of the high mean elevation of the terrain photographed and the greater distances between landing fields.

Area Coverage.—Table 3(*d*) indicates the estimated area coverage obtained on the various projects. The effect of power of the plane and the higher productivity obtained when flying over sectionized country are illustrated in the following:

Description	Average horsepower: 180 hp	350 hp
Coverage per flight (sq miles):		
Over sectionized areas	212	248
Over nonsectionized areas	195	164
General average	205	233

In nonsectionized areas a heavy plane seems to be at a greater disadvantage than a lighter plane, although the advantage is reversed in sectionized areas. No information could be obtained to explain the poorer performance of heavier planes in nonsectionized areas.

Coverage Per Exposure.—The cameras in use for vertical aerial photography produce photographs 7 in. by 9 in. and 9 in. by 9 in. in size. The areas of the photographs are therefore 63 sq in. and 81 sq in., respectively, and show a ground area (scale of 1 to 20,000) of 6.28 sq miles and 8.07 sq miles, respectively. If an average overlap and sidelap of 60% and 30%, respectively, are required, and the plane can maintain the necessary elevation, the net progressive area per exposure is 1.75 sq miles for the 7-in. by 9-in. photographs, and 2.24 sq miles for the 9-in. by 9-in. photographs. Apparently, however, the net average per exposure is considerably less as Table 3(*e*) indicates.

The SCS reported that a study of a project of about 68,000 sq miles indicates that each photograph (7 in. by 9 in.) covers a net area of approximately 1.26 sq miles.

Table 3(*e*) shows that there is some difference, even as between divisions, in the coverage that can be expected per exposure. Some divisions have had a higher proportion of 9-in. by 9-in. cameras on their projects, which accounts for the greater coverage per exposure. It appears that 1.25 sq miles and 1.75 sq miles per exposure of 7-in. by 9-in. and 9-in. by 9-in. cameras, respectively, may be used for estimating purposes. The difference between the theoretical

and the actual area per exposure is accounted for largely by the fact that the specifications require the contractor to photograph more than the actual area of the project. Most contracts required that each county be flown as a separate subproject. Certain requirements were met—as, for example, photographing several miles beyond the county boundaries which increases the number of exposures normally necessary for the area of the county. Consequently, the area of photography is greater than the area of the county and can be calculated by plotting flight lines on the map.

Reflights.—Upon completion of the initial photography of a project, the contractor may find that some of the work does not meet the requirements of the specification; that is, there may be gaps, insufficient overlap or sidelap, or other defects. Reflights are then necessary to obtain complete coverage. Upon inspection by the contracting officer, some of the material may also be found unsatisfactory and additional reflights are necessary.

From the available material, the following appears to be the average reflights by AAA divisions:

Division	Percentage of reflights
East Central	20.6
Northeast	29.3
North Central	29.0
Southern	28.7
Western	23.4
Average	26.3

The allowance for reflying is difficult to estimate since it is largely a matter of efficiency on the part of the contractor and of inspection on the part of the contracting officer. Past performance of the contractor and of the contracting officer, where known, should be used in determining the average for the estimate.

Unit Costs

General Considerations.—Determination of unit costs in the field of aerial photography is a hazardous venture. Only the contractor knows the initial and operating costs of his plant and equipment; the performance and efficiency of his planes and personnel; and other pertinent costs such as insurance, taxes, etc. There are numerous competitive differences between contractors such as prevailing scales of pay, quality and speed of planes and equipment, ability to use plant and equipment on other work, and location of main laboratory from project area—all of which make it difficult to estimate the cost of a project with precision. Nevertheless, it is possible to approximate the reasonable cost of a particular project.

Value of Plane and Equipment.—A study of the planes used in the AAA 1938 aerial photographic program considering value, power, and age of planes is summarized in Table 5(*a*). Because the medium and heavy planes are often purchased secondhand and also because very often contractors use these planes on other work, a cost of \$7,500 seemed reasonable for the plane. Similarly, studies of camera and other equipment indicated \$1,500 as a more than reason-

able estimate. Many of the cameras had been in service since World War I. Assuming a 10-yr period of depreciation for both plane and equipment and 6% interest on the investment, a charge of $100 per month should be made. Hangar rentals in 1937 varied from $30 to $50 a month—$40 per month probably being a fair charge. Obviously, it is not generally feasible to keep

TABLE 5.—Cost of Aerial Photography

		(a) Average Capital Costs			(b) Operating Costs (Dollars per Hour)					
			Probable Cost				Insurance			
No. of planes	Average horse-power	Age (years)	New, in dollars	Dollars per horse-power	Fuel	Over-haul	Lia-bility	Com-pensa-tion	Over-head	Total oper-ating cost
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
10	145	1.3	4,950	33.00	2.18	1.09	1.50	1.25	0.90	6.92
20	300	5.8	8,860	29.50	4.50	2.25	2.00	1.25	1.50	11.50
27	435	8.5	13,100	30.10	6.56	3.27	2.50	1.25	2.04	15.62

planes working all year. The charge to projects should be on the basis of 10-month operation per year. The yearly charge for the plane and yearly hangar rental totals $1,680; and, if overhead is assumed at 15%, the total yearly charge is $1,932 or $193 per month on a 10-month operating basis.

Operation and Maintenance of Planes.—From such sparse information as could be assembled it seemed that, assuming gasoline to cost 25¢ per gal and oil to cost 35¢ per quart, a rule-of-the-thumb formula of 1.5¢ per hp-hr could be derived. A similar formula for estimating the cost of overhauling and inspecting equal to 0.75¢ per hp-hr was derived. This value may be used in estimating despite the fact that aerial photographic pilots do much of the work themselves during the large amount of waiting time. Insurance on plane and equipment and public liability is quite high; and, although it is not known how much contractors carry, studies indicate that a reasonable charge is $1.50 per flying hr for light planes, $2 per flying hr for medium planes, and $2.50 per flying hr for heavy planes. In addition, one must add compensation insurance while flying, which varies from state to state. For estimating purposes, $1.25 per flying hr is assumed (Washington, D. C., rate in 1938). The cost of operating a plane is summarized in Table 5(*b*), assuming overhead at 15%. The hourly operating costs shown in this table explain the tendency of contractors to purchase new light planes in recent years.

Salaries of Pilot and Photographer.—The salaries of flyers and aerial photographers are subject to larger fluctuation. Some companies, for example, may pay entirely upon coverage completed and accepted; some may pay a base salary plus so much per square mile of accepted photography; and others may pay on a straight salary basis. The sums to be used in estimating should range between $500 and $600 per crew of two per month. An average of $550 per month may be used for estimating purposes.

Although compensation insurance for employees, while flying, was included in the operating cost of the plane (see Table 5(*b*)), there is an additional charge

for employees while on the ground. This charge is also subject to large variations. Using 1938 rates for Washington, D. C., compensation insurance would amount to \$20 a month for the crew. The Social Security insurance rate in 1938 was 4% of the salary, or a total of \$22 per crew. Accordingly, \$600 a month for the salary of the crew may be used in estimating; and to this sum should be added overhead, assumed at 15%. The total allowance for this item will be \$690 per month.

Cost of Flying.—The cost of flying cannot be reduced to a square mile basis until the project area is known and the number of days available determined. For estimating, the time is divided into two parts—waiting time and flying time. The waiting-time cost per month is the sum of salaries for the crew of \$690 and the charges for the plane and hangar rental of \$193, a total of, say, \$885 for estimating purposes. Flying time is computed on a flight basis. In the North Central States, for example, the average time of flight is 4.46 hours, or 4.5 hours for all practical purposes. For a medium-size plane, the cost per flight will be about \$51.80 or, say, \$52 for estimating purposes.

Because of the vagaries of the weather, the contractor must allow for some contingency. The method suggested in this analysis is to add to the waiting-time cost the percentage of variation shown in Fig. 2 for the area under consideration.

Cost of Photography.—Efficient organization plays a large part in the cost of photography. Volume production is another important element of cost. The following analysis is based on continuous production of large quantities and is not applicable to intermittent or low-volume production.

TABLE 6.—COST OF PHOTOGRAPHIC MATERIAL

Material	Quantity	Man-hours	Dollars per Stated Quantity			
			Labor	Mate-rial	Over-head	Total
Film	One roll	4	2.52	27.50	10.00	40.02
Contact Prints, Single Weight:						
Commercial stock	100, 8 in. by 10 in.	4	2.52	2.70	1.74	6.96
Waterproof stock	100, 8 in. by 10 in.	4	2.52	10.00	4.17	16.69
Negatives	100, 20 in. by 24 in.	55	34.70	141.00	58.57	234.27
Contact prints	100, 20 in. by 24 in.	9.5	6.00	19.70	8.57	34.27

A survey by the U. S. Department of Labor of twenty-five firms engaged in aerial photography indicated that, in 1938, the median wage paid to workers was from $52\frac{1}{2}$¢ per hr to $57\frac{1}{2}$¢ per hr. Firms on the East Coast and West Coast paid about 75¢ per hr, in Texas about 40¢ per hr, and in the Middle West about 60¢ per hr. For the purpose of this study, 60¢ is used which, with compensation and Social Security insurances (1938), would amount to 63¢ per hr. Table 6 contains the cost of the usual photographic material, when the camera used takes a 7-in. by 9-in. photograph. Overhead is estimated at $33\frac{1}{3}$% of the cost of labor and materials.

A necessary requirement of aerial photography is a photo-index sheet which is a composite photograph of all the pictures taken, after all reflights have been made. Such a sheet makes identification simple and quick when a photo-

graph of a particular area is required. To make such an index, contact prints are stapled on boards in the order and position that the photographs were taken in the air and are then copied. The coverage on one of these sheets, from actual tests, averages 240 sq miles. Assuming 1.25 sq miles per contact print, 192 prints will be required per sheet. Moreover, the contractor will be required to make a contact print for each exposure he makes in the air and, consequently, reflights of 30% must be included. Laboratory wastage of about 5% is an additional necessary allowance. Accordingly, the number of contact prints to be made for each sheet is 262 at \$6.96 per hundred—or a total cost of \$18.25. Prints may be stapled at about 25 per man-hr at a cost, for 192 prints, of \$4.85. Assuming \$6.00 for board, staples, and overhead, each board will cost about \$29.10 before it is photographed. The negative will cost about \$2.35 and the contact print will cost about 35¢—or a total of \$31.80 per index sheet.

The usual contract stipulates the delivery, by the contractor, of the aerial photographic negatives, one set of waterproof contact prints, one set each of photo-index negatives and photo-index sheets, and one set of single-weight prints which had been used in making the photo-index sheets. The photographic cost per square mile may be estimated by assuming a 1,000 sq mile area for simplicity of calculation. A roll of film will cover 137.5 sq miles (1.25 sq miles per exposure times 110 exposures per roll). Therefore, 1,000 sq miles will require 7.27 rolls of film, or 9.5 rolls allowing for 30% reflights. The number of waterproof prints required will be $\frac{1{,}000}{1.25}$ or 800 net. Allowing 5% waste, 840 prints will be printed. Reflying is not included in this item as these prints are made last, after all reflights have been finished. The number of index sheets is obtained by dividing 1,000 sq miles by 240 sq miles or 4.17 sheets. The cost in dollars per 1,000 sq miles (\$650) may be itemized as shown in Table 7.

TABLE 7.—COST OF MATERIALS FOR 1,000 SQUARE MILES BY PHOTOGRAPHY

Item	Quantity	Unit cost[a]	Total cost
Rolls of film	9.5 each	\$40.02	\$380.19
Waterproof prints	840 each	0.167	140.28
Photo-index sheets	4.17 each	31.80	129.43
Total cost (dollars)	1,000 sq miles	0.65	\$649.90

[a] Dollars per unit of quantity listed.

In 1938, some contractors were using cameras taking 9-in. by 9-in. photographs, a size now generally required. The cost of photography is somewhat less. This size of film comes in rolls of 150 ft and longer, instead of 75 ft for the 7-in. by 9-in. camera. For 150 ft the man hours required for developing is about 50% more than that shown in Table 6. The material cost is 100% more. Including overhead, the cost per roll is \$78.37. Printing productivity for 9-in. by 9-in. contact prints is the same as that shown in Table 6. The material cost is 20% higher—that is, in proportion to the areas of the photographic papers. A 7-in. by 9-in. print is made on an 8-in. by 10-in. paper, whereas a 9-in. by 9-in. print is made on a 10-in. by 10-in. paper.

The number of exposures may be computed by dividing the total length of the film in inches by 10 in. For a 150-ft roll, 180 exposures, each covering

1.75 sq miles, will cover 315 sq miles. Consequently the cost of film per 1,000 sq miles, including 30% reflights, will be $321.32. Similarly, allowing 5% waste, only 600 waterproof prints will be required per 1,000 sq miles at a cost of $116.10. The coverage of a photo-index map of 240 sq miles remains unchanged except that, instead of 262 prints of the 7-in. by 9-in. size, 187 of the 9-in. by 9-in. size will be required. The cost per index sheet is $26.56, computed as shown herein. Consequently, the cost per 1,000 sq miles is $547.42, or on a square-mile basis, about 10¢ less than the cost of photography using a 7-in. by 9-in. camera (see Table 7).

Other Cost Considerations.—Studies indicated that the size of project was an important element in the total cost. Every effort was made to block out contiguous areas of fairly large sizes for incorporation into single projects. In certain regions, areas averaged from 15,000 sq miles to 20,000 sq miles per project, and in others no more than 5,000 sq miles or 6,000 sq miles were possible. Adequate competition was another important factor. This could be controlled by releasing projects when more than enough equipment seemed to be available. Uniform specifications were absolutely necessary if costs were to be kept down; and this was done as nearly as practicable. Finally, efforts were made to schedule work on a year-round basis for as many planes as possible so as to spread the costs of equipment and overhead over more units of work. For this reason, contracts for aerial photography in the south stipulated that planes could take off on, or about, June 1 but had to be returned by, or before, December 1. This provided additional equipment for projects in areas where summer conditions provided greater utilization. Because of the known effect on costs of these variables, aerial photographic projects were cleared through the various bureaus of the U. S. Department of Agriculture and often with some of the state governments so that needs were all consolidated and programmed. Moreover, when bids were received and evaluated, they were subject to rejection if they exceeded, appreciably, the engineer's estimate of the project. The efficacy of the administrative policy implementing research conclusions was demonstrated by the reduction in costs obtained.

Programming and Estimating Projects

With such information, it is possible to program and estimate aerial photographic projects on, say, a scale of 1 to 20,000. Two examples will be used (both in the Middle West) to show the effect on the costs by reason of weather conditions. Both projects are to begin on June 1 and end before December 1. Project A (17,000 sq miles) is in northern Missouri, extending across the state, and project B (10,400 sq miles) is in southern Indiana. Project A cuts across weather regions 7 and 8, and project B is all in region 7.

Time Required.—By inspection, in Table 2, the average number of days in project A that are 0.1 or less overcast is about 7.5 per month, with a variation of 25%. As the area is about equally divided in regions 7 and 8, the monthly percentages will be averaged in project A. In project B, the probable average number of days per month is about 5.6, with a variation of about 27%. The time available for both projects using one-plane crews is computed as shown

in Table 8. (For method of computing clear days available see Table 1 and supporting text.)

Although the average coverage per flight for the Middle West is 242 sq miles, the heavier plane in sectionized areas can cover 248 sq miles. For estimating purposes assume 240 sq miles per flight. The number of flights, including reflights of 30% for project A will be 92 and for project B, 56. The

TABLE 8.—EXAMPLES OF ALLOCATING AVAILABLE TIME

Month	(a) Project A; 17,000 Sq Miles in Northern Missouri			(b) Project B; 10,400 Sq Miles in Southern Indiana		
	Percent of average month	Clear Days Available		Percent of average month	Clear Days Available	
		Number	Cumulative[a]		Number	Cumulative[a]
June	84	6.3	6.3	80	4.4	4.4
July	110	8.2	14.5	93	5.0	9.4
August	112	8.4	22.9	100	5.6	15.0
September	125	9.4	32.3	125	7.0	22.0
October	148	11.1	43.4	162	9.1	31.1
November	109	8.2	51.6	118	6.6	37.7

[a] Cumulative days available to a one-plane crew. For a two-plane crew, multiply these values by 2.

time required for project A is 6 months and for project B, say, 5 months, two planes and crews being used for each project. The waiting time in project A is about 5⅓ months, but because of the size of project the delays caused by inspection of material would justify the employment of the crew for the full six months. Extra flights—probably 10% of the total would be equitable—for reconnaissance and getting on and off the projects at the start and finish of the work must be added to the number of flights. Thus, project A would require one hundred and one flights and project B, sixty-two flights. The estimated costs of the projects are as listed in Table 9.

TABLE 9.—COMPARATIVE ESTIMATES OF COSTS

No.	Item	Project A		Project B	
		Cost	Remarks	Cost	Remarks
1	Waiting time	$10,620	Six months[a]	$ 8,850	Five months[a]
2	Contingency	2,660	25% of item 1	2,390	27% of item 1
3	Flying time	5,250	101 flights[b]	3,220	62 flights[b]
4	Photography	11,050	17,000 sq miles[c]	6,760	10,400 sq miles[c]
5	Subtotal	$29,580		$21,220	
6	Profit	4,420	15%	3,180	15%
	Estimated Costs:				
7	Total	$34,000		$24,400	
8	Unit	$2.00	Per square mile	$2.35	Per square mile

[a] Two planes at $885 each. [b] At $52 each. [c] At 65¢ per sq mile, assuming camera takes 7-in. by 9-in. photographs.

Depending on the camera used in 1938, project A may cost from $1.90 to $2.00 per sq mile, whereas project B may cost from $2.25 to $2.35 per sq mile.

Sufficient information has been given to estimate costs for future programs. The average values of plane and equipment will probably not change appreciably. The costs of operating and maintaining a plane and of photographic material can be adjusted in accordance with the prevailing salary and wage rates and the costs of materials. The productivity of labor, discussed in the section, "Cost of Photography," has not been materially affected by new developments and may be used for current estimates. Since the size of the aerial photograph now generally used (1946) is 9 in. by 9 in., the cost of photographic materials should be estimated on that basis. For aerial photography of a different scale (other than 1 to 20,000) or which uses a camera with a longer or shorter focal length than $8\frac{1}{4}$ in., the foregoing information could be used but this would require considerable manipulation.

Results Obtained

Analyses of a number of projects led to the belief that a properly administered program should result in average costs of from $2 to $2.50 per sq mile. In fact, an established year-round program might well cost less than $2 per sq mile. This is important to consider, especially since aerial photography has such widespread usefulness even outside the field of engineering. The campaign to reduce the cost of aerial photography (the studies for which began late in 1937 and began to be implemented in the spring of 1938) was successful because of the increasing familiarity with the work to be done on the part of both contractors and contracting officers. Instructions were issued to all the technicians in the department acquainting them with the elements of programming and of costs so that the maximum economy could be obtained. The following paragraph is quoted from the instructions:

TABLE 10.—Reduction in Costs of Aerial Photography by Careful Planning and Programming

Description	1926–1937[a]	1938	1939	1940	1941
Area covered, in square miles	970,434	635,562	445,844	376,436	381,083
Total cost, in dollars	4,110,423	1,968,668	926,273	765,281	892,320
Unit cost, in dollars per square mile	4.24	3.09	2.07	2.03	2.34

[a] Dates are inclusive, and all dates are for the calendar year.

> "While quality of performance is most important, the cost of aerial photographic projects must be kept to a minimum. Proper programming is essential to economy of operations. Insofar as possible, programs should be planned on a 12-month basis, taking every possible advantage of the probable weather conditions. Areas should be large enough to permit economy of operation. Changes in specifications should be examined to determine how costs may be effected. And, lastly inspections of materials should be made promptly and fairly to reduce idle time and waste of materials."

The results obtained more than justified the efforts and expenditures in making the studies. The reduction in costs is shown in Table 10 which was prepared from data submitted by the U. S. Department of Agriculture.

Acknowledgment

Although the work was done under the direction of the writer, it could not have been accomplished without the invaluable assistance and cooperation of the technicians in the various divisions of the Agricultural Adjustment Administration, the Soil Conservation Service, the Forest Service, and other bureaus, and of some of the contractors who were generous of their time and ideas. The writer is particularly grateful for the aid which he received from Marshall S. Wright, Office of the Secretary; Louis A. Woodward, SCS; Grover M. Plew and J. Marvin Cultice, AAA, all of the U. S. Department of Agriculture.

Conclusions

The studies reported in this paper need greater testing and refinement, because the usefulness of aerial photography is just beginning to be understood and appreciated. Cultural changes in some parts of the United States were so rapid, even before the beginning of World War II, that it is more economical to refly the areas than to use old photography. For proper planning, many areas will require reflying every 5 years, on an average. When the use of aerial photography becomes widespread (as it may in the not-too-distant future) flying and photographing of from 400,000 to 500,000 sq miles per yr may be a normal operation. When that time comes more precise information may need to be developed.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
REPORTS

Engineering Education

PROGRESS REPORTS OF THE SOCIETY'S PROFESSIONAL COMMITTEE ON ENGINEERING EDUCATION, FOR 1944 AND 1945

1944 COMMITTEE

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WILLIAM J. ARMENTO
FRANKLIN THOMAS
(Contact Member)

JANUARY 18, 1946

CONTENTS

NOTE: Studies of engineering education have been pursued by the Society's committee for a number of years. The report for 1944 received private distribution and comment. That for 1945 suggested wider attention. The Board of Direction at its January, 1946, meeting approved and authorized the publication of both reports.

[These reports, as given herein, are presented for information and for comment to headquarters, either for printing in *Proceedings* or for transmitting direct to the committee. Comments on the similar work of the Society for the Promotion of Engineering Education are not to be included.—Ed.]

ENGINEERING EDUCATION

PROGRESS REPORT OF THE SOCIETY'S PROFESSIONAL COMMITTEE ON ENGINEERING EDUCATION FOR 1944

The data presented in this study have been secured for the most part from the catalogs of institutions having accredited civil engineering curricula. Inasmuch as the detailed requirements for graduation stated in the catalogs are in some cases a little difficult to interpret, it is not thought that the data represent exactly the procedures followed in the institutions considered. The committee is sure, however, that for all practical purposes the values shown in the various tables are close enough to the exact values so that conclusions may be drawn therefrom without serious error.

Statistical data are given in Table 1 which is a summary of reports and in Table 2 which is a more detailed analysis.

Digest of Data

Foreign Language Requirements.—Considering the curricula of 114 schools it was found that only six required a foreign language for the bachelor's degree; and one of these will permit the substitution of history for a foreign language in case the student has had two years of the latter study in high school. It should be noted that a considerable number of engineering schools require two years of foreign language in high school.

Humanistic-Social Studies.—Nearly one half of the engineering colleges now devote 18% or more of the student's time to free electives, English, and economics.

Physical Sciences Including Geology.—Fifteen schools of the 114 require less than 12% of the total time to be devoted to these subjects; fifty three of the colleges devote between 14% and 17%; and sixty four allot between 14% and 18% of the educational time to this group.

Engineering Drawing and Descriptive Geometry.—Fifty one of the schools devote between 4% and 5% of the educational time to the study of these subjects, and sixty five of the schools employ between 4% and 6% of the time in such a manner.

Mathematics.—Ninety five of the schools use between 10% and 15% of the educational time in the study of mathematics.

Mechanics, Hydraulics, and Strength of Materials.—Fifty nine of the schools devote between 10% and 13% of the time to the study of these subjects. Only three schools devote more than 13% to such studies.

Other Engineering—Elpctrical, Mechanical, Etc.—Seventy one of the schools devote between 5% and 11% of the time to other engineering subjects; twenty

TABLE 1.—Percentages of Total Hours Required for the Bachelor of Science Degree in Civil Engineering

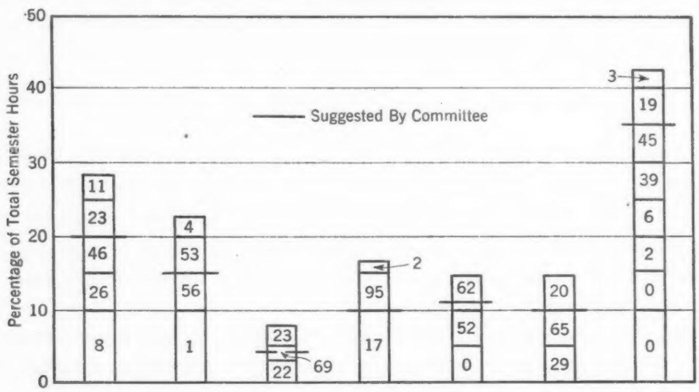
Explanation

Institutions are ranked in ascending order within each subdivision of subject matter. Therefore the position of an institution will not be the same in each subdivision. For instance, institution 20 in mathematics might be institution 30 in "Other Engineering."

The requirements for the Bachelor of Science degree are frequently expressed in terms in semester hours. Institution A may require 138 semester hours; institution B, 144. A three semester-hour course is one which meets three times a week over a period of one semester. Lecture and recitation classes occupy approximately one hour at each meeting of the class. Laboratory classes rather generally occupy three hours at each class meeting. A laboratory class meeting once a week for a three-hour session would carry one semester-hour credit.

A considerable number of institutions are on the quarter rather than the semester basis. Their requirements for graduation are expressed in quarter hours instead of semester hours. By multiplying by two thirds, the former may be expressed in terms of the latter.

The numerals in bars of the graph give the number of schools within the bracketed percentages. For example, thirty-nine schools spend from 5% to 30% of the students' time on civil engineering subjects.



Institutional rank	Free Electives (English, social science) (1)	Physical science, including geology (2)	Drawing (3)	Mathematics (4)	Mechanics, hydraulics, and strength of materials (5)	Other engineering (6)	Civil engineering (7)	Institutional rank
1	6.6	9.5	1.4	6.3	6.3	0.0	17.3	1
2	6.9	10.3	2.2	6.7	6.3	0.0	18.4	2
3	7.9	10.4	2.4	6.8	6.6	0.0	20.6	3
4	7.9	10.6	2.5	7.3	6.9	1.9	22.6	4
5	8.0	11.0	2.5	7.9	7.2	1.9	23.3	5
6	8.4	11.3	2.5	8.0	7.3	2.8	24.4	6
7	9.7	11.3	2.6	8.2	7.4	2.9	24.4	7
8	9.8	11.3	2.6	8.4	7.6	2.9	24.6	8
9	10.5	11.4	2.6	9.0	7.6	2.9	25.0	9
10	10.6	11.5	2.7	9.1	8.3	3.2	25.0	10
11	10.9	11.7	2.7	9.2	8.3	3.3	25.0	11
12	11.1	11.8	2.7	9.3	8.4	3.7	25.1	12
13	11.9	11.9	2.7	9.5	8.5	3.8	25.2	13
14	12.2	11.9	2.7	9.5	8.5	4.2	25.2	14
15	12.2	11.9	2.8	9.6	8.5	4.2	25.4	15
16	12.3	12.1	2.8	9.7	8.5	4.2	25.4	16
17	12.4	12.3	2.8	9.8	8.5	4.3	25.6	17
18	12.4	12.4	2.8	10.0	8.6	4.3	25.7	18
19	12.5	12.6	2.8	10.1	8.7	4.4	25.7	19
20	12.5	12.8	2.9	10.3	8.7	4.5	26.0	20

TABLE I.—(*Continued*)

Institutional rank	Free Electives (English, social science) (1)	Physical science, including geology (2)	Drawing (3)	Mathematics (4)	Mechanics, hydraulics, and strength of materials (5)	Other engineering (6)	Civil engineering (7)	Institutional rank
21	12.5	13.0	2.9	10.4	8.7	4.6	26.0	21
22	12.8	13.0	2.9	10.4	8.8	4.6	26.0	22
23	12.8	13.2	3.0	10.4	8.9	4.6	26.2	23
24	12.9	13.2	3.1	10.6	9.0	4.6	26.4	24
25	13.2	13.2	3.2	10.6	9.1	4.7	26.4	25
26	13.3	13.2	3.3	10.7	9.1	4.7	26.4	26
27	13.4	13.2	3.4	10.8	9.2	4.8	26.5	27
28	13.4	13.4	3.7	10.9	9.3	4.8	26.5	28
29	13.6	13.4	3.7	10.9	9.3	4.9	26.5	29
30	13.8	13.5	3.7	10.9	9.3	5.0	26.7	30
31	14.4	13.5	3.7	11.0	9.3	5.0	27.1	31
32	14.6	13.5	3.8	11.0	9.4	5.0	27.3	32
33	14.6	13.6	3.8	11.0	9.4	5.0	27.6	33
34	14.7	13.7	3.8	11.0	9.4	5.1	27.9	34
35	15.0	13.7	3.8	11.0	9.4	5.2	28.2	35
36	15.1	13.8	3.9	11.1	9.5	5.2	28.3	36
37	15.1	13.8	3.9	11.1	9.5	5.3	28.3	37
38	15.1	13.8	3.9	11.1	9.5	5.5	28.4	38
39	15.2	13.8	3.9	11.1	9.6	5.5	28.4	39
40	15.3	13.8	3.9	11.1	9.6	5.5	28.5	40
41	15.3	13.9	4.0	11.3	9.6	5.7	28.6	41
42	15.4	14.0	4.0	11.4	9.7	5.7	28.6	42
43	15.6	14.1	4.0	11.4	9.7	5.8	28.6	43
44	15.7	14.1	4.0	11.4	9.7	5.8	29.3	44
45	15.7	14.2	4.0	11.4	9.7	5.9	29.4	45
46	15.8	14.3	4.0	11.5	9.7	5.9	29.7	46
47	16.1	14.3	4.1	11.6	9.7	6.0	29.9	47
48	16.1	14.4	4.1	11.6	9.7	6.0	30.1	48
49	16.3	14.4	4.1	11.7	9.7	6.0	30.1	49
50	16.3	14.5	4.1	11.8	9.8	6.1	30.2	50
51	16.5	14.5	4.1	11.8	9.8	6.1	30.3	51
52	16.5	14.6	4.1	11.8	9.9	6.1	30.5	52
53	16.6	14.7	4.1	11.8	10.0	6.1	30.6	53
54	16.6	14.8	4.1	11.8	10.0	6.2	30.6	54
55	16.6	14.8	4.1	11.8	10.0	6.2	30.7	55
56	16.6	14.8	4.2	11.9	10.0	6.3	30.9	56
57	16.7	14.9	4.2	12.0	10.0	6.3	31.0	57
58	16.7	15.0	4.2	12.0	10.1	6.3	31.0	58
59	16.8	15.0	4.2	12.0	10.1	6.4	31.1	59
60	17.1	15.0	4.2	12.1	10.2	6.6	31.2	60
61	17.3	15.0	4.2	12.2	10.2	6.6	31.2	61
62	17.4	15.0	4.2	12.2	10.2	6.7	31.8	62
63	17.7	15.1	4.2	12.3	10.3	6.8	31.9	63
64	17.8	15.1	4.2	12.3	10.3	6.9	31.9	64
65	18.0	15.2	4.3	12.3	10.3	7.0	32.0	65
66	18.2	15.2	4.3	12.4	10.4	7.1	32.0	66
67	18.3	15.3	4.3	12.4	10.4	7.1	32.0	67
68	18.3	15.3	4.3	12.4	10.6	7.2	32.1	68
69	18.4	15.3	4.3	12.5	10.6	7.2	32.2	69
70	18.5	15.3	4.3	12.5	10.7	7.3	32.4	70
71	18.6	15.5	4.3	12.5	10.7	7.3	32.4	71
72	18.8	15.5	4.3	12.5	10.8	7.3	32.5	72
73	19.0	15.6	4.4	12.5	10.8	7.4	32.5	73
74	19.2	15.7	4.4	12.5	10.8	7.4	32.5	74
75	19.4	15.8	4.5	12.5	11.0	7.5	32.6	75
76	19.5	15.8	4.5	12.5	11.0	7.6	32.7	76
77	19.7	15.9	4.5	12.5	11.1	7.7	32.8	77
78	19.7	15.9	4.5	12.5	11.1	7.8	32.8	78
79	19.8	16.0	4.5	12.5	11.1	7.8	32.8	79
80	19.9	16.0	4.6	12.7	11.2	8.0	33.1	80
81	20.0	16.1	4.6	12.7	11.2	8.2	33.2	81
82	20.0	16.3	4.6	12.7	11.3	8.5	33.2	82
83	20.0	16.3	4.7	12.7	11.3	8.5	33.3	83
84	20.6	16.4	4.8	12.7	11.4	8.7	33.5	84
85	20.7	16.5	4.8	12.7	11.4	8.9	33.6	85
86	20.7	16.6	4.8	12.7	11.4	9.0	33.8	86
87	20.7	16.6	4.8	12.7	11.4	9.0	34.0	87
88	20.8	16.6	4.8	12.8	11.5	9.0	34.0	88
89	20.8	16.6	4.9	12.8	11.5	9.0	34.4	89
90	20.9	16.6	4.9	12.8	11.5	9.3	34.5	90

TABLE I.—(*Continued*)

Institutional rank	Free Electives (English social science) (1)	Physical science, including geology (2)	Drawing (3)	Mathematics (4)	Mechanics, hydraulics, and strength of materials (5)	Other engineering (6)	Civil engineering (7)	Institutional rank
91	21.1	16.6	4.9	12.9	11.6	9.4	34.6	91
92	21.4	16.7	5.0	12.9	11.6	9.5	34.9	92
93	21.8	16.7	5.1	12.9	11.6	9.6	35.0	93
94	21.8	16.7	5.2	13.0	11.8	9.7	35.2	94
95	21.9	17.0	5.3	13.0	11.8	10.0	35.2	95
96	22.4	17.0	5.4	13.0	11.8	10.1	35.3	96
97	22.4	17.1	5.5	13.2	11.8	10.2	35.6	97
98	22.6	17.2	5.5	13.2	11.9	10.4	35.7	98
99	22.9	17.3	5.6	13.3	11.9	10.4	35.8	99
100	23.0	17.3	5.6	13.5	11.9	10.9	36.0	100
101	23.6	17.3	5.6	13.5	12.0	11.1	36.0	101
102	24.2	17.4	5.6	13.6	12.0	11.1	36.2	102
103	24.9	17.8	5.6	13.6	12.0	11.1	36.6	103
104	25.1	17.8	5.6	13.6	12.1	11.2	37.4	104
105	25.1	17.9	5.6	13.8	12.1	11.6	37.6	105
106	25.5	18.0	6.1	13.9	12.1	11.8	37.8	106
107	25.7	18.0	6.2	14.3	12.1	11.9	38.0	107
108	26.2	18.6	6.3	14.4	12.2	12.3	38.2	108
109	26.4	19.3	6.5	14.5	12.3	12.7	38.5	109
110	26.4	19.3	6.8	14.8	12.4	13.2	38.8	110
111	27.0	21.2	6.9	14.8	12.4	13.3	38.8	111
112	27.4	21.3	7.1	14.8	14.0	13.4	40.6	112
113	27.8	21.5	7.4	15.3	14.7	14.4	40.9	113
114	28.1	22.4	7.6	16.7	14.7	14.7	42.3	114

nine schools devote less than 5%; and thirteen of the schools devote less than 4%.

Civil Engineering Subjects.—The percentage of strictly civil engineering subjects in civil engineering curricula varies within rather wide limits. For instance, in one institution 17.3% of the student's educational time is required for this purpose. At the other extreme one institution requires 42.3% in such subjects. Two of the more widely known engineering colleges of the United States require, respectively, 40.6% and 40.9%. Sixty four of the schools devote between 30% and 40% of the time to strictly civil engineering subjects; and forty seven devote less than 30%. In the case of two schools, this percentage is less than 20%. A study made by the Co-operative Committee on Civil Engineering Education of the Society for the Promotion of Engineering Education (SPEE) shows a very wide variation in subject matter in the civil engineering subjects taught in the various curricula.

Suggestions

1. The committee suggests that the Society encourage engineering schools to devote 20% of the time of civil engineering curricula to the study of humanistic-social subjects; and, in cases where it is feasible, to have the subjects coordinated so that the student may have a rather thorough acquaintance with one or more areas within these fields.

TABLE 2.—Detailed Analysis, Percentage of Total Hours Required for the Bachelor of Science Degree in Civil Engineering

Explanation

(*a*) *Free Electives, English, and Economics.*—Inasmuch as the Committee on Engineering Education After the War of the Society for the Promotion of Engineering Education (SPEE) has recommended that a minimum "of approximately 20 per cent of the students' educational time" be devoted to the humanistic-social studies, an analysis of the time given to such studies in 114 institutions should be of interest. Similar data for the other types of subjects are also given. Thirty four of the 114 schools are now devoting 20% or more of the educational time to free electives, English, and economics. Forty two of the schools are spending 19% or more of the educational time in this type of work. These schools could easily comply with the recommendation of the SPEE committee with regard to the humanistic-social stem so far as time requirements are concerned.

(*b*) *Other Engineering* (*Electrical, Mechanical, Etc.*).—Seventy one of the schools devote between 4.9% and 11% of the time to the study of other engineering subjects, such as electrical and mechanical engineering.

(*c*) *Physical Sciences, Including Geology.*—Fifty three of the colleges devote between 13.9% and 17% of the educational time to the study of the physical sciences, including geology.

(*d*) *Drawing* (*Engineering Drawing and Descriptive Geometry*).—Fifty one of the schools devote between 3.9% and 5.0% of the educational time to the study of engineering drawing and descriptive geometry, plus, in some cases, engineering mechanism.

(*e*) *Mathematics.*—Ninety five of the schools devote between 9.8% and 15% of the educational time to the study of mathematics.

(*f*) *Mechanics, Hydraulics, and Strength of Materials.*—Fifty nine of the schools devote between 9.9% and 12.5% of the educational time to the study of mechanics, hydraulics, and strength of materials.

(*g*) *Civil Engineering.*—Forty five of the schools devote between 29.9% and 35% of the time to the study of strictly civil engineering subjects. It should be noted that forty seven of the schools devote less than 30% of the time to this specialization, and that in the case of two schools the percentage is less than 20%.

(*a*) Free Electives, English, and Economics

Number of institutions (1)	Percentage of total time (2)
4	8.0
8	10.00
13	12.00
30	14.00
46	16.00
64	18.00
72	19.00
80	20.00
90	21.00
95	22.00
99	23.00
101	24.00
103	25.00
107	26.00
110	27.00
113	28.00
114	28.10

(*b*) Other Engineering (Electrical and Mechanical, Etc.)

Number of institutions (1)	Percentage of total time (2)
3	0.0
5	2.0
13	4.0
29	5.0
46	6.0
64	7.0
79	8.0
85	9.0
94	10.0
100	11.0
107	12.0
109	13.0
112	14.0
114	15.0

(*c*) Physical Sciences, Including Geology

Number of institutions (1)	Percentage of total time (2)
1	10
4	11
15	12
20	13
41	14
57	15
78	16
94	17
105	18
108	19
110	21
113	22
114	23

(*d*) Drawing (Engineering Drawing and Descriptive Geometry)

Number of institutions (1)	Percentage of total time (2)
1	2.0
22	3.0
40	4.0
91	5.0
105	6.0
111	7.0
114	7.6

(*e*) Mathematics

Number of institutions (1)	Percentage of total time (2)
3	7
5	8
8	9
17	10
30	11
56	12
93	13
106	14
112	15
113	16
114	17

(*f*) Mechanics, Hydraulics, and Strength of Materials

Number of institutions (1)	Percentage of total time (2)
4	7.0
9	8.0
23	9.0
52	10.0
74	11.0
100	12.0
111	13.0
111	14.0
114	15.0

(*g*) Civil Engineering

Number of institutions (1)	Percentage of total time (2)
1	18.0
2	19.0
2	20.0
3	21.0
4	23.0
5	24.0
8	25.0
19	26.0
30	27.0
34	28.0
43	29.0
47	30.0
56	31.0
64	32.0
79	33.0
86	34.0
92	35.0
99	36.0
103	37.0
106	38.0
111	39.0
111	40.0
113	41.0
114	42.4

2. It is suggested that the civil engineering curriculum may reasonably be subdivided into subject groups, so far as time is concerned, approximately as follows:

Subject	%
Humanistic-social	20
Physical sciences, including geology	15
Drawing	4
Mathematics, not including trigonometry	10
Mechanics, hydraulics, and strength of materials	11
Engineering subjects, other than civil	10
Civil engineering	30
Total	100

Discussion of Suggestions.—To provide space in the curriculum for certain of the subjects contained in these suggestions, some reduction or replacement of time allocations to presently taught subjects may be required. It will become evident and must be accepted that not all the subjects which would be useful and beneficial to a prospective civil engineer can be accommodated within a four-year curriculum. Some choices between desired material must be made, and some reductions of time for specific subjects will be necessary to achieve the balanced programs represented by the suggestions as to subject distribution. To achieve the broadened curriculum it may be necessary to reduce the time devoted to subjects which would be classified under the headings of practice and technique. Time devoted to professional applications and undergraduate specialization may also have to be re-evaluated. Preference as to retention of professional subjects should be given to those of an analytical nature rather than to those of descriptive character.

In evaluating subjects to determine their places in the curriculum, it is well to consider their usefulness as a means of developing the "engineering method" in the mind of the student as well as the content of the courses.

In the selection of subjects for retention in the curriculum, it is important to identify them as fundamental or functional. The fundamental subjects can be acquired out of college only with difficulty, if at all; although knowledge or skills pertaining to practice or technique, beyond a brief introduction, can generally be learned to better advantage by experience on the job.

So long as the bachelor's degree is continued in America as the reward for successfully completing a four-year curriculum, it certainly is a recognition which may reasonably be expected by anyone completing four years of as rigorous a program as an engineering curriculum, even though it is no longer possible to compress within such a time limit an introduction to all of the expanding field of knowledge which is comprised within the scope of civil engineering.

Practicing engineers generally concede that it is desirable for engineering graduates to have the benefit of some instruction in liberal arts courses. There are two patterns of curricula which might be followed to accomplish this desirable objective:

(1) To superimpose a professional training upon a formal liberal arts foundation as is done in the fields of medicine and law; and

(2) To intermingle the humanistic subjects with the scientific and engineering subjects in an undergraduate curriculum with the recognition that one or more years of graduate work would be required to accommodate advanced professional subjects for which time was not available in the undergraduate program.

Until there is a general adoption among the engineering colleges in the United States of the first pattern of training, it is futile, as has been demonstrated in a few instances, for individual institutions to require that the engineering training be preceded by several years of liberal arts. Prospective engineering students do not select such "delayed action" curricula when others are available which enable them to enter immediately upon their engineering training; nor is it conclusive that the first pattern is preferable or would produce equal results. In the second pattern there would tend to be greater selectivity in the advanced years with more definite assurance that ability, rather than economic circumstances, would determine the opportunities for continuing in the later years. There are nearly always graduate scholarships or fellowships available for the really able students.

Report of SPEE Committee on Engineering Education After the War

No discussion of the problems of engineering education at this time would be complete without a consideration of the SPEE report in the May, 1944, issue of the *Journal of Engineering Education.* The report is founded to some extent on the 1940 Report on Aims and Scope of Engineering Curricula made by a committee of the SPEE.

The Society's committee endorses with some rather limited qualifications this 1944 report of the SPEE Committee on Engineering Education After the War. Discussion on several points of the report follow.

(a) The report advocates programs for three major groups of students as follows:

1. Those who would follow engineering programs of the usual pattern. This group would continue to comprise a majority of undergraduate engineering students.
2. Those preparing for careers in the operation and management of industry.
3. Those who would be fitted for unusual scientific and creative accomplishments.

In discussing this proposal the report says,

"For all of the programs mentioned, we believe that the first three years should conform to customary patterns, modified in general characteristics as suggested in this report, and with variations in subject matter of physical and engineering science in accord with the several major branches of en-

gineering. After the third year, however, more fundamental differentiation would begin. For the first two groups—regular students and the industrial group—professional subject matter should be different in nature and functional in purpose. The industrial group would give major attention to matters relating to production and operation, while the regular group would concentrate on scientific-technological studies. The content of the humanistic-social studies would be the same for both groups as would be the total duration, generally four academic years, required for the baccalaureate degree.

"The program for the group preparing for highly scientific and creative engineering work should be differentiated from the other programs not later than the end of the junior year, in order to provide broader and more fundamental preparation in scientific principles and methods than is needed for the general run of students. This program would of necessity be longer in duration than the others and would extend from the end of the junior year through one or more years of graduate study leading to the master's or doctor's degree * * *."

As regards this recommendation, the committee does not consider it feasible to determine at the end of the third year the particular group into which the student would best fit, to predict at that time with any degree of certainty those who are to be the future engineers and scientific specialists, those who are to manage and operate, and those who are to follow industrial work. A four-year curriculum in civil engineering should be essentially the same for all students with limited provision for specialization in the senior year. Those desiring further specialization can secure it through graduate work in college, on the job, in extension classes, or in evening school.

(b) The report advocates a humanistic-social stem of studies running throughout the four years and occupying at least 20% of the student's time. Emphasis is laid on the necessity of coordinating this work so that at the end of college life the individual will have had more than a course here and a course there without any connection between them.

As to this point, the curriculum studies of the committee show that 30% of the curricula examined are now devoting 20% or more of the student's time to free electives, English, and economics; and that 36.8% of the curricula devote 19% or more of the time to such studies. It seems entirely feasible for the remaining schools to adopt this time allotment if they so desire.

(c) In regard to technical institutes, the SPEE committee states:

"* * * one step that appears to be of primary and immediate importance in this connection, especially because of the impending complications of post-war years, is the professional means of formal recognition for institutes offering programs of the technical institute type and for their graduates * * *."

Also:

"* * * we believe that industry and the engineering profession as well as the schools and colleges should give united support to measures of recognition of sound technical institute programs, and of certification of their graduates for a considerable range of technical pursuits."

With these sentiments the committee is in close accord. However, the committee does not feel that the creation of a large number of new technical institutes would be of value either to concerns employing civil engineers or to the profession. A multiplicity of such institutes would open another means of access into the civil engineering profession whereby the qualifications of the persons concerned, academically speaking, would be of a much lower grade than they are at the present time. Furthermore, there does not appear to be as great a demand in civil engineering for persons possessing this type of training as there is in the other main divisions of the engineering field.

The position of the technical institute in engineering education is not well defined. There are no clear-cut boundaries between the vocational school, the technical institute, and the engineering college. In some quarters it is felt that no need exists for the technical institute because enough men to fill subprofessional jobs are supplied by those who fail to complete the full college programs or by in-service training of men with only elementary education.

In other quarters it is felt that neither the vocational school nor the engineering college can provide the desired training of technicians in industry.

In view of this uncertainty, the committee feels that it is necessary for the Society to keep alert to developments in the technical institute programs; to be prepared to cooperate in measures designed to guide developments in their proper channels; and to assist such groups as the New York State Association of Engineering Colleges in insisting that adequate study be given to the need for such institutes before any are authorized by state or federal action.

The committee believes that subprofessional jobs in civil engineering must not be preempted by an oversupply of technical institute graduates. These jobs give the necessary preliminary experience and training to recent graduates of engineering schools (engineers-in-training) on the road to more responsible professional positions. It would not be in the public interest to block this road.

ENGINEERING EDUCATION

PROGRESS REPORT OF THE SOCIETY'S PROFESSIONAL COMMITTEE ON ENGINEERING EDUCATION FOR 1945

During the current year the committee has devoted its attention to the making of a survey of membership opinion relative to the educational needs of civil engineers. Early in the summer 2,700 copies of a questionnaire (see Appendix) designed by the Cooperative Committee in Civil Engineering Education of the Society for the Promotion of Engineering Education (SPEE) were sent to a list of members of the Society prepared by the secretaries of the Local Sections.

Of these 2,700 copies: 1,035 completed questionnaires were returned to the committee; or 7.3% of the corporate membership recorded in the 1945 Yearbook. Active interest in the subject of education is demonstrated by the fact that 38% of those receiving the rather long questionnaire took the time to complete it.

The questionnaire furnished personal data concerning the answerer, and his opinions relative to (1) the effectiveness of civil engineering graduates of the last decade, (2) the importance of basic sciences and engineering subjects other than civil, (3) civil engineering subjects, and (4) other subjects. The statistical data gathered from the questionnaire are presented in the Appendix.

Deficiencies of Graduates: Recommendations of the 1944 Report

Majority opinion rated the graduates of the past ten years as poor in ability (1) to write a clear, orderly letter or report, (2) to address logically and effectively, either a private group or a public gathering. Also, majority opinion accused these graduates of showing little interest in public affairs.

The following suggestions of the 1944 report of this committee should be of interest in connection with these criticisms, particularly with regard to the last one:

1. The committee suggests that the Society encourage engineering schools to devote 20% of the time of civil engineering curricula to the study of humanistic-social subjects; and, in cases where it is feasible, to have the subjects coordinated so that the student may have a rather thorough acquaintance with one or more areas within these fields.

2. It is suggested that the civil engineering curriculum may reasonably be subdivided into subject groups, so far as time is concerned, approximately as follows:

Subject	%
Humanistic-social	20
Physical sciences, including geology	15
Drawing	4
Mathematics, not including trigonometry	10
Mechanics, hydraulics, and strength of materials	11
Engineering subjects, other than civil	10
Civil engineering	30
Total	100

Opinions Expressed in Questionnaire

Effectiveness of Civil Engineering Graduates.—Graduates of the last decade are rated as follows: Average or above average in ability (1) to apply mathematics to practical problems, (2) to apply engineering fundamentals to practical problems, and (3) to analyze a problem formulated by others. They possess only average ability to formulate a problem for themselves; they are rather poor in understanding the limitations of formulas and in the ability to consider all cost factors.

The graduate's ability to write a clear, orderly letter or report is considered poor. His ability to address, logically and effectively, either a private group or a public gathering is rated as very poor.

Generally speaking, little fault is found with his ability to get along with people, his capacity for leadership, and his grasp of basic fundamentals.

He is rated high as far as accuracy, fairness, diligence, and dependability are concerned.

Although a considerable percentage of graduates are interested in public affairs, a much larger group, according to this survey, have but little interest in such matters.

Replies to the questionnaire indicate that the placing of more emphasis on nontechnical courses and less emphasis on specialized engineering courses, would be of benefit to the influence and prestige of civil engineers. However, the vote was about evenly divided on the question as to whether or not such a procedure would adversely affect the professional competence of the civil engineer.

Importance of Basic Sciences and Engineering Subjects Other than Civil.—Practically all concede that freshman mathematics is of great importance; the differential and integral calculus is considered of great importance or of moderate importance (evenly divided) by the great majority, although 10% thought it of little importance; differential equations rates rather low.

Physics took a high place; general chemistry is regarded highly or of moderate importance except by about 11% of the voters who express the opinion that it is of little importance.

There is no doubt in the mind of anyone about the importance of mechanical drawing and descriptive geometry; freehand drawing takes a secondary place.

Machine shop, as a part of the civil engineering curriculum, is not looked on with favor.

Under the heading of engineering mechanics, the following subjects stand at the top: Statics, dynamics, strength of materials, and materials testing laboratory. Somewhat less than half the graduates feel that aerodynamics is of moderate importance; the remainder are divided between little and great importance in the ratio of more than 2 to 1.

Hydraulics ranks high; fluid mechanics falls into a secondary place in the opinion of the majority.

Engineering geology and elementary electrical engineering fall between great and moderate importance.

Elementary machine design, thermodynamics, and elementary internal combustion engines are in the position of having more people indicate that they are of little importance than of great importance, although in each case a majority of voters expresses the opinion that these subjects were of moderate importance.

One real surprise was found in the case of statistical analysis. More than 30% of the voters indicated that the subject was of great importance, and only 15% relegated it to the third position.

Civil Engineering Subjects.—The following subjects are awarded, and with emphasis, a position of great importance: Reinforced concrete, statically determinate structures, foundations, structural design (steel), plane surveying, engineering contracts and specifications, construction engineering, structural design (timber), soil mechanics (theory), public water supply, highways, sewerage and sewage disposal, statically indeterminate theory, and masonry structures.

Voted a place of moderate importance by the majority and a place of first importance by a considerable percentage of the returns are: Hydrology, irrigation, and drainage, water power engineering, valuation and appraisal, city planning, and railroads.

The following were considered of moderate importance by the majority but with a substantial percentage expressing the opinion that they were of little importance: Advanced surveying (including photogrammetry), shop detailing, river and harbor engineering, tunneling, airplane structures, geodetic surveying.

Rating of Other Subjects.—Appraised for their value as a part of an educational background for a civil engineer, those subjects designated as of great importance are: Report writing, English composition, public speaking, professional relations, and engineering economy.

Other subjects appraised as belonging somewhere between moderate importance and great importance are: Personnel and labor relations, American government, public administration, psychology, business law, economic theory, ethics, finance, and logic.

The following subjects rate between moderate importance and little importance, although in each case a number expressed the opinion that they were of primary importance: History, industrial management, geography, literature, industrial history, modern languages, accounting, international relations, philosophy, sociology, music appreciation, patents, and art.

The subject of a foreign language appears higher on the list than it really should, because blank votes are not counted under the heading of little importance. There were many blank votes on this subject. Furthermore, many who did express an opinion voted for the practical use of the language rather than for its value as a part of an educational background.

Conclusions and Recommendations.—

1. The committee believes that the suggestions incorporated in the 1944 Report of the Committee on Engineering Education relating to the subdivision of time in the four-year curriculum are reinforced by the returns from the survey of membership opinion. It is possible to devote approximately 20% of the time to nontechnical subjects and yet to include the fundamental technical and science content in the four-year curriculum leading to the degree of Bachelor of Civil Engineering.

2. The returns from the survey emphasize again the great importance of courses in English composition, report writing, and public speaking. Since the importance of such courses has been known for years to engineering college administrators it might be well to suggest to them a thorough reexamination of course content, teaching methods, and standards of performance in these courses with a view to attaining all possible improvement in the abilities of the engineering graduate and to alleviate the criticism of the engineers because of shortcomings in these respects.

3. Some of the questionnaires returned indicate that the answers were influenced by the geographical location and the specialized experience of the respondent. The great majority indicated however that fundamental courses rather than specialized applications are of the most importance in undergraduate curricula.

4. Special attention is directed to the importance attached to the subject of professional relations. Most colleges have no specific course in this subject; nevertheless, the practicing engineer believes that the college student should receive some training in this field. Engineering teachers should be encouraged to explore what can be done to instil a better knowledge of professional relations in the young engineer.

5. The committee respectfully directs the attention of the Board of Direction to the 1944 report and recommends its adoption and publication together with this 1945 report. The 1944 report was referred by the Board to the deans and presidents of engineering schools for their comments. Practically all those who replied commented favorably on the report. Only one reply was critical of the entire report and one other of a part. The committee believes that there are many data in both the 1944 and 1945 reports which will be of particular value to engineering teaching staffs currently engaged in curriculum studies and revision. Moreover, many of those who answered the questionnaire this year and supplied curricula data last year have requested information of the results of the surveys.

6. The committee believes that the replies (see Fig. 3 in the Appendix) of the questionnaire (opinions of the effectiveness of civil engineering graduates) constitute in some degree an indictment of engineering teaching. The committee

had planned to initiate this year a study of the qualifications of the civil engineering teacher but, after some consideration, felt that the study should be deferred until next year. The subject is one of great importance and the committee recommends that the 1946 committee be requested to undertake this study, possibly in cooperation with the Committee on Civil Engineering Education of the Society for the Promotion of Engineering Education. The study should seek to determine how engineering teaching may be improved and in what ways the Society can cooperate in achieving this objective.

APPENDIX

1945 Questionnaire on Engineering Education

In this Appendix are reported details of the 1945 Questionnaire, in order as follows: Letter of transmittal (Fig. 1); personal data re answerer (Fig. 2); and specific questions with tabulated answers (Fig. 3, Table 3, and Table 4).

AMERICAN SOCIETY OF CIVIL ENGINEERS
33 West 39th Street
New York 18, N. Y.

Dear Sir:

Your opinion is desired on the vital subject of civil engineering education at one of the most important periods in educational history. The accelerated trends, technical and social, place new responsibilities upon the profession and upon civil engineering education. The importance of the problem and the Society's stake in the future justify the effort of obtaining the opinions of practicing engineers and employers at this time on the matters covered by the attached questionnaire.

The questionnaire is framed in an effort to determine the fundamental educational background deemed most helpful to the professional career of *all civil engineers.* In the course of events most practicing engineers become specialists in one or several fields. This survey, however, is seeking the core of fundamentals, technical and nontechnical, which constitutes the common educational base for entering all fields of civil engineering specialization. Engineering educators can plan the details of this basic education, but the viewpoints of practicing engineers and employers concerning the qualifications desired in the graduate will be most helpful. Engineering schools emerging from the war, in many cases without a student body, have an exceptionally fine opportunity to rebuild their curricula.

This is an opportunity wherein an expression of your judgment will have consideration at a particularly significant time.

Yours very truly,

IVAN C. CRAWFORD
The ASCE Committee on Engineering Education,
Cooperating with the Cooperative Committee on
Civil Engineering Education of S.P.E.E.

Fig. 1.—Letter of Transmittal

In an effort to obtain some idea of (1) how well trained the graduates were, and (2) the relative importance of the courses named in the questionnaire, as reflected in the tabulation, opinions have been arbitrarily weighed as good, average, and poor corresponding to great, moderate, and little importance.

The weight factor has been multiplied by the number of corresponding replies and the products thus obtained have been divided by the total number of replies to obtain the rating.

Thus, in Fig. 3, general opinions for "Ability to apply mathematics to practical problems":

$$\text{Rating} = \frac{3\,(437) + 2\,(431) + (1)\,68}{437 + 431 + 68} = 2.39$$

AMERICAN SOCIETY OF CIVIL ENGINEERS

Survey to Determine the Fundamental Educational Needs of All Civil Engineers

by

The Committee on Engineering Education of the

American Society of Civil Engineers

Cooperating with

The Cooperative Committee on Civil Engineering Education of S.P.E.E.

PART A. Information Relative to Person Answering Questionnaire

Name.. Age...........

Address..

Grade of Membership in American Society of Civil Engineers (Please Check)

Member........... Associate Member........... Junior........... Non-Member...........

Registered Engineer: Yes........... No...........

Major Field of Interest or Employment During Past 10 Years

						Others (Designate)
Airports	....	Highways	....	Railroads	....	....
City Planning	....	Hydraulics	....	Sanitary	....	....
Construction	....	Irrigation	....	Surveying & Mapping	....	....
Education	....	Power	....			
Engineering Economics	....	Public Utilities	....	Structural	....	....
Foundations	....			Waterways	....	....

General Status in Field of Work

Employer.................. Administrator or Manager.................. Consultant..................

Duties largely Technical................ Other..

Curriculum Studies in College

C. E............ Ch. E............ E. E............ M. E............ Other..

Degrees..

Non-Graduate............ Years in College..........

In requesting your opinions on the following pages [Fig. 3 and Table 4], it is intended that they apply to the content of the curriculum which includes in its scope basic sciences and engineering subjects other than civil; civil engineering subjects; and other or general subjects.

FIG. 2.—EXAMPLE OF SURVEY FORM

(1) Please rate the effectiveness of the civil engineering graduates of the last decade with respect to the following attributes:

	Good	Average	Poor
Ability to apply mathematics to practical problems	437	431	68
Ability to apply engineering fundamentals to practical problems	379	498	68
Ability to analyze a problem formulated by others	355	497	64
Ability to formulate a problem for themselves	159	539	220
Ability to understand the limitations of formulas	123	523	280
Ability to consider all cost factors	67	446	404
Ability to write a clear, orderly letter or report	53	364	518
Ability to address, logically and effectively, either a private group or a public gathering	36	191	704
Ability to get along with people	347	530	50
What is their capacity for leadership?	161	601	189
How is their grasp of basic fundamentals?	294	538	78
How would you rate their accuracy?	400	487	44
How would you rate their thoroughness?	307	531	78
How would you rate their diligence?	482	404	43
How would you rate their dependability?	529	363	27
How would you rate their interest in public affairs?	92	341	502

(2) If colleges place more emphasis on nontechnical courses and less emphasis than formerly on specialized engineering courses, would the effect upon the influence and prestige of civil engineers be favorable or unfavorable?

Favorable 712 Unfavorable 251

(3) If colleges place more emphasis on nontechnical courses and less emphasis than formerly on specialized engineering courses, would the effect upon the professional competence of the civil engineer be favorable or unfavorable?

Favorable 456 Unfavorable 487

Remarks:

FIG. 3.—GENERAL OPINIONS (SEE TABLE 3)

TABLE 3.—Ratings of Subjects Taught to Civil Engineers

Item (1)	Courses (2)	Rating (3)	Item (1)	Courses (2)	Rating (3)
	(*a*) General Opinions			(*b*) Basic Sciences	
1	How would you rate dependability?	2.55	1	Unified match	2.98
2	How would you rate diligence?	2.47	2	Strength of materials	2.89
3	Ability to apply mathematics to practical problems?	2.39	3	Physics	2.84
4	How would you rate accuracy?	2.38	4	Mechanics (statics)	2.83
5	Ability to apply engineering fundamentals to practical problems?	2.33	5	Hydraulics	2.71
6	Ability to get along with people?	2.32	6	Mechanics (dynamics)	2.71
7	Ability to analyze a problem formulated by others?	2.32	7	Mechanical drawing	2.59
8	How would you rate thoroughness?	2.25	8	Materials testing laboratory	2.47
9	How is grasp of basic fundamentals?	2.23	9	Calculus	2.36
10	What is capacity for leadership?	1.97	10	Engineering geology	2.34
11	Ability to formulate a problem?	1.93	11	General chemistry	2.32
12	Ability to understand the limitations of formulas?	1.86	12	Descriptive geometry	2.32
13	Ability to consider all cost factors?	1.63	13	Elementary electrical engineering	2.21
14	How would you rate interest in public affairs?	1.56	14	Statistical analysis	2.18
15	Ability to write a clear, orderly letter or report?	1.50	15	Freehand lettering	2.03
16	Ability to address, logically and effectively, either a private group or a public gathering	1.28	16	Fluid mechanics (other than water)	1.94
			17	Thermodynamics	1.85
			18	Elementary machine design	1.84
			19	Aerodynamics	1.79
			20	Elementary internal combustion engines	1.73
			21	Machine shop	1.72
			22	Advanced mathematics (differential equations, etc.)	1.60
	(*c*) Other Subjects			(*d*) Civil Engineering Subjects	
1	Report writing	2.86	1	Reinforced concrete	2.79
2	English composition	2.85	2	Statically determinate structures	2.77
3	Public speaking	2.72	3	Foundations	2.76
4	Professional relations	2.57	4	Structural design (steel)	2.74
5	Engineering economy	2.44	5	Plane surveying	2.69
6	Personnel and labor relations	2.30	6	Engineering contracts and specifications	2.66
7	American government	2.24	7	Construction engineering	2.61
8	Public administration	2.22	8	Structural design (timber)	2.57
9	Psychology	2.20	9	Soil mechanics (theory)	2.48
10	Business law	2.19	10	Indeterminate structures	2.48
11	Economic theory	2.14	11	Public water supply	2.40
12	Ethics	2.08	12	Highways	2.38
13	Finance	2.04	13	Sewerage and sewage disposal	2.33
14	Logic	2.01	14	Masonry structures	2.32
15	History	1.95	15	Soil mechanics (laboratory)	2.20
16	Industrial management	1.93	16	Hydrology	2.14
17	Geography	1.92	17	Irrigation and drainage	2.11
18	English literature	1.88	18	Water power engineering	2.10
19	Industrial history	1.85	19	Valuation and appraisal	2.07
20	Modern languages	1.85	20	City planning	2.04
21	Accounting	1.80	21	Airports	2.03
22	International relations	1.79	22	Railroads	2.03
23	Philosophy	1.71	23	Advanced survey, including photogrammetry	1.93
24	Sociology	1.69	24	Shop detailing	1.89
25	Music appreciation	1.34	25	River and harbor engineering	1.84
26	Patents	1.28	26	Tunneling	1.77
27	Art	1.21	27	Airplane structures	1.76
			28	Geodetic surveying	1.63

TABLE 4.—Subject Content of Curriculum

Please indicate by a check in the appropriate column, your opinion as to whether each subject listed is of great importance, moderate importance, or of little importance in the college training of all civil engineers. Please base your decision upon the significance of the training and subject matter in these fields, irrespective of your opinion of any particular course which you may have had. In evaluating the subjects in Table 4(*c*), please take particular care to appraise the value of training and fundamental knowledge in the field represented by the title of the subject. Each subject field is to be appraised for its value as part of an educational background for a civil engineer.

(*a*) Basic Sciences and Engineering Subjects Other than Civil (See Table 3(*b*))

Course	Great importance	Moderate importance	Little importance
Mathematics:			
Elementary (algebra, trigonometry, analytical geometry)	916	80	0
Differential and integral calculus	469	470	97
Advanced (differential equations, etc.)	114	378	513
General chemistry	412	495	89
Physics	864	150	6
Engineering Drawing:			
Mechanical	645	319	50
Freehand	298	415	270
Descriptive geometry	458	424	42
Machine shop	112	491	393
Engineering Mechanics:			
Statics	677	134	18
Dynamics	743	247	23
Strength of materials	921	110	2
Materials testing laboratory	584	376	91
Aerodynamics	146	461	346
Hydraulics	737	257	16
Fluid mechanics (gases and liquids other than water)	197	546	253
Engineering geology	412	536	64
Elementary electrical engineering	315	545	131
Elementary machine design	185	480	350
Thermodynamics	169	518	317
Elementary internal combustion engines	121	491	412
Statistical analysis	351	473	175
Other (describe)			

(*b*) Civil Engineering Subjects (See Table 3(*d*))

Course	Great importance	Moderate importance	Little importance
Surveying:			
Plane	661	239	24
Advanced including photogrammetry	208	519	275
Geodetic	91	441	459
Highways	461	489	69
Railroads	248	542	221
Tunneling	149	475	376
Structures:			
Statically determinate theory	796	176	27
Statically indeterminate theory	549	373	72
Design (steel)	766	236	14
Design (timber)	614	350	44
Reinforced concrete	824	185	13
Masonry structures	462	414	141
Airplane structures	151	433	389
Shop detailing	213	481	320
Foundations	766	210	16
Soil Mechanics:			
Theory	538	438	47
Laboratory	350	522	147
Irrigation and drainage	279	675	159
Water power engineering	264	574	166
Hydrology	300	519	159
Public water supply	485	417	92
Sewerage and sewage disposal	485	420	141
River and harbor engineering	166	522	324
City planning	265	530	221
Airports	241	530	211
Construction engineering (estimates and costs; planning and plant; management)	660	286	54
Engineering contracts and specifications	698	261	42
Valuation and appraisal	278	464	208
Other (describe)			

TABLE 4.—*Continued*

(*c*) Other Subjects (See Table 3(*c*))

Course	Great importance	Moderate importance	Little importance
English composition	893	132	10
Report writing	894	130	9
Public speaking	841	269	22
Business law	299	583	114
Industrial management	188	566	255
Personnel and labor relations	374	470	168
Economic theory	312	510	174
Engineering economy	517	426	71
Finance	229	591	191
Accounting	152	502	356
Patents	22	239	736
Professional relations (duties and obligations, professional ethics, social responsibilities of the engineers, etc.)	651	289	72
Psychology (fundamentals of human behavior)	380	440	183
English literature	206	496	327
History	214	540	269
Industrial history	163	549	307
Geography	223	473	298
Political Science:			
American government	382	493	136
Public administration	366	486	148
International relations	146	499	360
Philosophy	142	407	455
Logic	326	370	318
Ethics	352	399	267
Sociology	123	433	424
Modern language (designate)	163	369	281
Art	28	342	578
Music appreciation	25	267	646
Others (designate)			

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

FLOW FROM DRAINAGE BASINS DETERMINED BY SHORT-TERM RECORDS

Discussion

BY LLOYD L. HARROLD

LLOYD L. HARROLD,[12] ASSOC. M. AM. SOC. C. E.[12a]—In view of the limited discussion of this paper, the writer could possibly infer that the procedures used therein have the sanction of the profession; but he will not. Although certain parts of the method received no comment—either confirmatory or refutatory—some phases of the procedure have been used for analyzing short-term watershed runoff for a number of locations. The writer has been reluctant, however, to adjust any of the runoff curves to normalcy. This hesitancy was in full view of the fact that almost without exception the observed rainfall rates for the periods of record have exceeded the normal expectancy. Long-term records at near-by U. S. Weather Bureau Stations, or Yarnell values, were used as normal in this comparison. The adjustment, if used, would lengthen the period of recurrence in each case; or, for the same recurrence interval, would reduce the flood peak. The unadjusted curves give flood-peak values that may be high. It appears better to have this factor of safety in the design flood peak rather than to apply an adjustment that may be questionable.

Mr. Roberts' preference for log-log paper and the use of Eq. 3 (the reciprocal of Eq. 1) does not introduce any material change in the flood values for 25, 30, or 100 years. Data taken from the unadjusted curves in either Fig. 4 or Fig. 8 would be almost identical. Even the plotted rainfall points in both figures compare similarly with the Yarnell 5-min curves—Mr. Roberts' conclusion to the contrary. The plotting method does not eliminate the adjustment to normalcy, if such an adjustment is warranted in the first place.

Both the writer and Mr. Roberts feel the inadequancy of defining flood-frequency curves from short records. This has been brought out rather forcibly. A 6-yr record, 1939–1944, for a 122-acre watershed had been analyzed at the end of 1944 (Fig. 9(*a*)). The rainfall intensities and the soil moisture

NOTE.—This paper by Lloyd L. Harrold was published in April, 1945, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1945, by Samuel Roberts.

[12] Project Supervisor, SCS, North Appalachian Experimental Watershed, Coshocton, Ohio.

[12a] Received by the Secretary January 21, 1946.

conditions recorded in this 6-yr period appeared to establish the fact that a normal period had been sampled. Press of other duties prevented the completion and publication of the report on this analysis. In view of the 1945 records this delay proved to be a blessing. It is necessary, frequently, to pre-

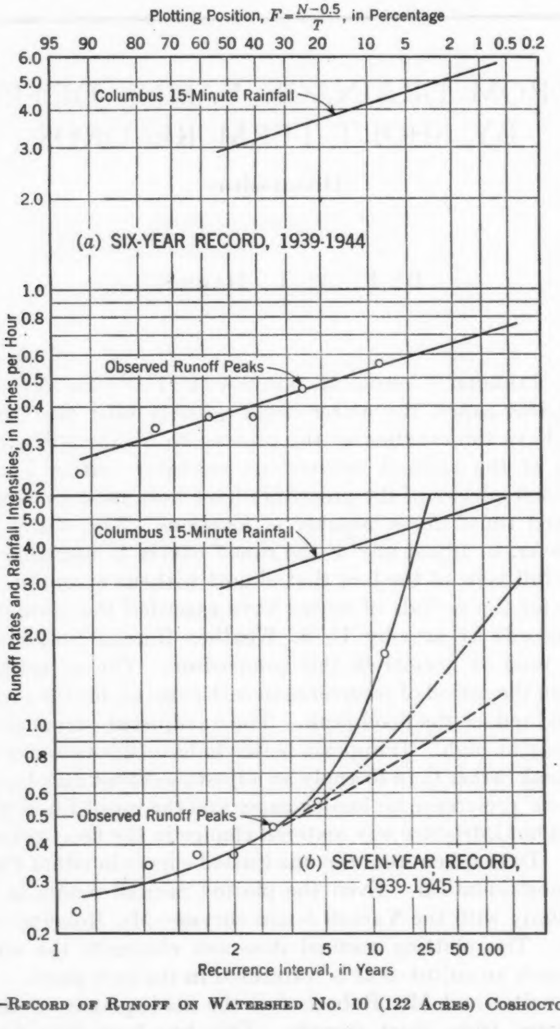

Fig. 9.—Record of Runoff on Watershed No. 10 (122 Acres) Coshocton, Ohio

pare preliminary reports which may be used until sufficient amount of additional data are obtained to warrant a revision on preparation of a final report. One cannot continue, indefinitely, to delay the analyses and publication of such reports.

In September, 1945, a flood peak of 1.72 in. per hr was recorded—three times as great as the highest flood in 6 years. The maximum 15-min rainfall

for the storm was not exceedingly high—3.56 in. per hr. This is about a 5-yr storm according to the late David L. Yarnell, M. Am. Soc. C. E. Antecedent moisture, however, was very high, but not more than that occurring in 1941 (date of highest peak prior to 1945). Six inches of rain fell in the 3-week period prior to the date of the 1945 storm. There was no material change in the land use over the watershed throughout the entire 7-yr period. From an extension of the curve in Fig. 9, the recurrence interval of this 1945 peak would be somewhere above 1,000 years. This is unreasonable.

The peak values for the 7-yr period, 1939–1945, were replotted as shown in Fig. 9(*b*). The recorded peak values for the 6-yr and 7-yr periods are listed in Table 4. A line through the plotted points defines curve A, Fig. 9(*b*)). An extension of this curve crosses the rainfall curve. At a 50-yr recurrence the flood peak would be about twice as large as the rainfall rate. This is most unreasonable. Mr. Roberts' statement, that the frequency curve need not be the average of all the plotted points, certainly applies to this condition. All the points should not be used in defining this curve.

TABLE 4.—FLOOD PEAKS, WATERSHED NO. 10

Order	SIX-YEAR RECORD (1939–1944)		SEVEN-YEAR RECORD (1939–1945)	
	Runoff (in. per hr)	Plotting position	Runoff (in. per hr)	Plotting position
1	0.56	8.3	1.76	7.2
2	0.46	25	0.56	21
3	0.37	42	0.46	36
4	0.37	58	0.37	50
5	0.34	75	0.37	64
6	0.24	92	0.34	79
7	0.23	...	0.24	93
8	0.207	...	0.23	...
9	0.198	...	0.207	...
10	0.192	...	0.198	...
11	0.146	...	0.192	...

A curve (line C, Fig. 9(*b*)) through all but this highest value gives the 1945 peak as one occurring once in a little less than 1,000 years. This seems to be too great. Obviously, the probable recurrence of this peak value is somewhere between curves A and C—perhaps along curve B. Only a longer period of record will help to establish the true position of this flood-frequency curve.

Mr. Roberts' interest in this paper and his discussion are appreciated. The writer is subjecting the procedures to further study and may at some future date submit his findings to the profession.

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DISCUSSIONS

ELIMINATION OF CROSS CONNECTIONS IN LOS ANGELES, CALIF.

Discussion

BY HARRY HAYES

HARRY HAYES,[11] ASSOC. M. AM. SOC. C. E.[11a]—Mr. Van Meter's indictment of the apathy of public officials in prosecuting cross-connection control is no longer applicable to the City of Los Angeles. There has been such apathy in the past, however; but, after a change in personnel and an intensive program of education, an active spirit of cooperation has become evident among all city officials concerned with the elimination of cross connections. Several of these officials have undertaken aggressive programs in their respective spheres of influence.

The Army post mentioned by Mr. Lowe is not a new installation, but a permanent post constructed many years ago. Almost all the plumbing fixtures were old in style and constituted dangerous cross connections. The water system was heavily overloaded and partial vacuums were a common occurrence. These conditions, together with many cross connections of long standing, created a hazardous health problem.

Since 1945, there has been a notable addition to the cross-connection control program in the City of Los Angeles—the establishment of a $50,000 Foundation for Cross-Connection Control and Research by the University of Southern California, conducted under the direction of the dean of engineering. A separate laboratory building has been constructed and equipped under the auspices of this foundation for the complete testing of all backflow-prevention devices, from the smallest size, to and including 12-in. pipe sizes. This Foundation began its work by conducting rigorous research into all conditions of cross-connection control and backflow-prevention devices, and it is anticipated that the findings of this body will constitute a source of impartial and authoritative information on all such matters. In addition, a plumbing laboratory has been established to determine standards for backflow devices used in connection with standard plumbing.

NOTE.—This paper by Harry Hayes was published in March, 1945, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1945, by Roy O. Van Meter; and October, 1945, by Robert P. Lowe.

[11] Asst. San. Engr., Dept. of Water & Power, Los Angeles, Calif.

[11a] Received by the Secretary January 14, 1946.

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DISCUSSIONS

ECONOMICS OF PHOTOGRAMMETRY

Discussion

By Oliver S. Reading, and Leon T. Eliel

Oliver S. Reading,[11] M. Am. Soc. C.E.[11a]—An excellent description of present photogrammetric practices and of their varied applications and limitations is presented in this paper. Comparisons of the relative economy or efficiency of the different instruments and combinations of methods emphasize the great flexibility of photogrammetry. As Mr. Eliel has shown, excellent work may be done by almost any combination of methods.

Many factors in addition to the methods described in the paper cause wide variations in efficiency. The character of the terrain, size and location of project, skill of personnel, quality of photographs—any of these factors are capable of causing a greater range in cost than the variation in efficiency of the methods used. Although many of these factors will average out as cost data are accumulated for a very large number of projects, fully adequate data for determining costs are not yet available. To the present time most photogrammetric work has been so sporadic and subject to such fluctuation of funds and to such urgency for production regardless of cost, training, or skill when funds have been available, that present cost data probably are misleading.

Arguments based on theoretical considerations are likely to err both from lack of consideration of factors which may have been unimportant on projects with which the individual is familiar, and from differences in evaluation resulting from special conditions that may prevail in certain organizations. Thus, the factor of smaller scale and larger area in air photographs is limited by the ceiling of the aircraft and the resolution of the lenses and photographic materials available. The ratio of control cost to number of photographs is important, but this argument favors extrawide-angle, or multi-lens photographs requiring different instruments. Also, where many roads and adequate breakdown of geodetic control exist, the cost of supplemental control may become a

Note.—This paper by Leon T. Eliel was published in March, 1945, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1945, by Raymond A. Hill, Raymond L. Moore and Ralph S. Pearse; October, 1945, by George D. Whitmore, Benjamin A. Wasil, Charles H. Davey, and Robert H. Randall; and December, 1945, by W. J. Ryan.

[11] Chf., Research Section, Div. of Photogrammetry, U. S. Coast and Geodetic Survey, Washington, D.C.

[11a] Received by the Secretary December 31, 1945.

minor consideration. Again, if it were easier and faster to trace, accurately, the lines of a map by the manipulation of wheels moving the tool in the X-direction with one hand and in the Y-direction with the other hand than to guide the tool directly by hand, probably the wheel drives would be widely used for engraving. However, more information on this question will be available soon from a direct comparison of two plotting machines under construction in the photogrammetric office of the United States Coast and Geodetic Survey. One of the machines will use direct motion of the hand, accompanied by motion of the head in tracing the lines of the map; and the other will drive the tracing point by an X-handwheel and a Y-handwheel, while the operator's head remains stationary as with the stereoplanigraph.

Until recently there has been but one stereoplanigraph in the United States, and it has been operated much of the time by a man who admittedly has exceptionally acute stereoscopic vision and skill as an operator. The capture and importation of several stereoplanigraphs from Germany to the United States will allow a direct comparison in the same office, on the same projects, of the different machines and methods. Thus, much more reliable data should be obtained on the most economical field of application for each instrument or combination. Such direct comparisons of the stereoplanigraph, wide-angle multiplex, and 9-lens stereoplotters are contemplated in the Division of Photogrammetry of the U. S. Coast and Geodetic Survey during 1946 and 1947. Other organizations also will doubtless make direct comparisons.

Incidentally, recent surveys indicate that multi-lens cameras and equipment are more efficient than the 95° single-lens equipment for many projects. Multi-lens photographs are far from obsolete, although possibly further development of extrawide-angle lenses with admitted distortion in the air photograph corrected in subsequent printing, or of lenses using spherical focal surfaces may lessen the advantages of multi-lens cameras.

Certainly, photogrammetry has arrived at maturity in that no sizable mapping project can be executed with maximum efficiency unless some use is made of photographic surveys; yet those who are concerned with improving the efficiency and usefulness of photogrammetry are eagerly trying so many promising ideas (new lenses, camera stabilization, simpler plotters, color film, and radar control) that the economics of photogrammetry will surely require frequent discussion in the future.

LEON T. ELIEL,[12] ESQ.[12a]—It is very gratifying to the writer that his paper provoked so much excellent and constructive discussion. He is especially grateful that all the discussion on such a controversial subject has been presented with great temperance and restraint.

The twelve stereoplanigraphs captured from Germany should yield some fine comparative data between the different photogrammetric systems, as suggested by Mr. Reading. It is to be hoped that they will actually produce very efficiently—which objective certainly transcends, in importance, all academic considerations.

[12] Vice-Pres., Fairchild Aerial Surveys, Inc., Los Angeles, Calif.

[12a] Received by the Secretary February 4, 1946.

Among the discussions were six by map makers and two by map users. The latter, by Messrs. Hill and Ryan, have been especially constructive. Mr. Hill's remarks are of special importance to users who are less interested in how aerial photography is done, than they are in what it can do.

Mr. Ryan has given a thoughtful and thorough analysis of the application of photogrammetric methods to the problems of the forest engineer. The difficulties of control, inability to see the ground through the trees, and the problems of weather apply almost universally, as well as to forest problems.

Frequent reference has been made to the advantage the multiplex enjoys in "bridging" and thus minimizing control. Reports presented to the American Society of Photogrammetry show that the Germans (who invented and use both the stereoplanigraph and the multiplex) when under the pressure of war preferred the stereoplanigraph to the multiplex for bridging, using the multiplex to fill in the details.

The multiplex gives about half the vertical accuracy obtainable from the stereoplanigraph when the flight altitude is the same. For comparable accuracy, in other words, multiplex pictures are taken from half the height required for stereoplanigraph flying. In the bridging method, the accuracy is found to be somewhat less than if control had been available for each model. Assuming that a "bridge" of a certain length gives half the accuracy obtainable from control in every model, the altitude for stereoplanigraph flying and a "bridged" multiplex series would be 4 to 1. Thus, the stereoplanigraph area per model, compared to the "bridged" multiplex area per model, at comparable vertical accuracy, might be 16 to 1.

In this hypothetical case, bridging causes such a sacrifice in flight altitude that the "bridge" would have to be sixteen models long to have the same control economy as the stereoplanigraph with control in each model. Whether the bridge involves sixteen models, or some greater or smaller number, it is apparent that there is a diminishing return that may actually prove that the stereoplanigraph requires less control. Add the inefficiency of handling all the additional models required with the lower flight altitude and the "bridging" argument can readily "boomerang" in favor of the stereoplanigraph.

Probably W. Schermerhorn of the Dutch University at Delft, Holland, and Prime Minister of Holland, has conducted more research on bridging than any one. He concluded that, if bridging must be done, the stereoplanigraph was better for the purpose than the multiplex. He operated both instruments, as well as many others, very extensively. Of course, his experiments were with the Zeiss narrow-angle multiplex which Americans who have used both consider inferior to the Bausch and Lomb's wide-angle instrument.

The writer sees little justification for bridging as an economy measure. Only under war conditions or when the terrain is inaccessible because of natural conditions does there seem to be a clean-cut case for bridging; and, if bridging must be done, many authorities prefer the stereoplanigraph to the multiplex.

Little information was available on the K.E.K. plotter or the Reading 9-lens plotter, when this paper was first written. Fortunately, these new instruments have been described in the discussion.

The writer recommends a "decent burial" for one plotting instrument—the aerocartograph, which was mentioned in one discussion as comparable to the stereoplanigraph. Having had the two instruments side by side for twelve years, the writer considers that they are alike only in complexity of appearance and nomenclature. In fact, the aerocartograph is so completely obsolete that sometime soon the writer will be glad to offer a good one to a museum.

Mr. Davey made it very clear that, for small-scale mapping, the airplane cannot fly high enough to exceed the efficient limits of any of the methods. Thus, for 50-ft contours the multiplex flying could be from 30,000 ft aboveground, whereas the stereoplanigraph flying should theoretically be done at 62,500 ft. The latter is obviously impossible as 30,000 ft is substantially the limit of presently available aircraft. Thus, the contour interval must be less than 50 ft before the efficiency of the stereoplanigraph can be realized.

In summary, no photogrammetric method is a cure-all for mapping problems. Photogrammetry is a new and useful tool, and it richly supplements all the other well-known methods. Under many conditions, photogrammetric methods are superior. Sometimes the methods are not practicable, and yet again, a combination of methods is best. In discussing photogrammetry, engineers should not lose sight of the many valuable applications of aerial photography. A great many more square miles are studied usefully from aerial photographs and mosaics than from contour maps.

Each of these new aerial tools should be placed carefully in its proper category of usefulness.

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DISCUSSIONS

STRUCTURAL SKEW PLATES

Discussion

BY WILLIAM A. CONWELL, AND JULIUS SIGMAN

WILLIAM A. CONWELL,[23] M. AM. SOC. C. E.[23a]—In cataloging and demonstrating the various methods available for the solution of plate problems, the author has performed a remarkable service to the profession. In particular, the presentation of the method for applying difference equations to a determination of the bending moments in a plate, whether skew or rectilinear, will be appreciated by most engineers, who prefer physical concepts to involved mathematics. The paper will serve, without doubt, as a text for future study by those engineers who have long been disturbed by rule-of-thumb, semi-theoretical, and semi-empirical methods and have sought a practical analytical approach.

The writer wishes to call attention to some of the properties of the square network as shown in Fig. 3(*a*) and Eqs. 21*a* and 21*b*. The simplicity of the equations becomes more apparent when the equations are solved for $\bar{w}_k$ and $\bar{z}_k$, respectively. The values of $\bar{w}_k$ and $\bar{z}_k$ are then evidently merely the averages of the four adjacent values in the network, adjusted by one fourth of the quantity appearing on the right-hand side of Eqs. 21*a* and 21*b*.

This property of simplicity permits the use of numerical methods[24] in the solution of the problem. Reasonable values for $\bar{w}$ and $\bar{z}$ are first assumed for each nodal point on the network and then adjusted by the application of the equations to each point in turn. When a value thus computed differs from the assumed or previously computed value, it is substituted for the assumed or previously computed value and used in all the computations which follow until another application of the equation results in a further adjustment. Each assumed value is thus adjusted and readjusted until the equations are satisfied at each point. For large networks, this process would be a laborious one if it

NOTE.—This paper by F. L. Ehasz was published in June, 1945, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1945, by John W. Allen.

[23] Pittsburgh, Pa.

[23a] Received by the Secretary November 20, 1945.

[24] "A Method of Obtaining Moments in a Slab by the Numerical Solution of Differential Equations," by William A. Conwell, thesis submitted to the Carnegie Inst. of Technology, Pittsburgh, Pa., in 1937 in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering.

were not for the fairly quick convergence which may be secured by a method outlined by George H. Shortley, Royal Weller, and Bernard Fried.[25]

There is an old engineering practice of using an even number for the last digit in a dividend when the division results in a fraction of a half. Used strictly in numerical procedures, this practice assures accuracy in the results even though only a few significant figures are used in the computations. Because of this fact, elaborate machine computations are avoided and the calculations are reduced to simple arithmetical procedures and slide-rule work. Furthermore, it is possible to obtain values rapidly, which are correct to two significant figures, for rough designs or estimates. The use of such values is fully justified by the fact that the values represent a considerable improvement over values obtained from rule-of-thumb or pseudo-analytical processes. In its final form, furthermore, a slab or plate does not ordinarily fulfil all the assumed conditions of homogeneity, elasticity, "isotropicity," uniformity of thickness, etc.; and, hence, more elaborate computations may be warranted only for detailed analysis or research.

In addition to its simplicity, the square network can be applied universally to either rectilinear plates or those having any degree of skew. The hexagonal network, as demonstrated by the author, is convenient in the solution of plates having a 30° skew. Not only is the square network the natural one to use for a 45° skew but, again because of its simplicity, it is a good choice for any skew. Its application results in a solution in which two irregular edges are substituted for two straight-line edges; but, as the writer will demonstrate, this has little effect on the final results. Values adjacent to the irregular edge are in error, of course; but, for an equal-sided symmetrical plate, this error presents no problem as accurate values are available along the straight edge, symmetrical to the irregular edge. For other than symmetrical, equal-sided plates two solutions may be necessary—one with the network based on one of the edges and another with the network based on the edge that was irregular in the first solution. The solution of such plates by other than the square network would also result in a considerably more complicated procedure than that required for symmetrical equal-sided plates.

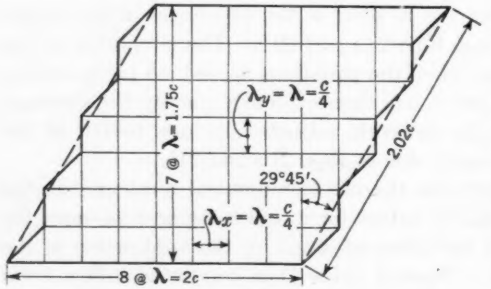

FIG. 9.—SQUARE NETWORK FOR THE SOLUTION OF A 30° SKEW PLATE WITH EQUAL SIDES

To demonstrate the foregoing statements, the writer has prepared a solution of the simply supported, equal-sided 30° skew plate supporting a uniform load. Fig. 9 shows the square network used in the solution, with its irregular edges and its span of 1.75 c (as against the actual 1.73 c) in the direction of Y—with

[25] "Numerical Solution of La Place's and Poisson's Equations," by George H. Shortley, Royal Weller, and Bernard Fried, *Bulletin No. 107*, Eng. Experiment Station, Ohio State Univ., Columbus, September, 1940.

the resulting discrepancies of 15 min in skew and 0.02 c in the length of the irregular edges. That these irregularities do not influence results beyond the accuracy required for engineering design is evident from comparison of the results with those of the author's sixteen-coefficient solution, as shown in Table 8. Inasmuch as the network used did not have a nodal point at the

TABLE 8.—COMPARISON OF MOMENTS AND DEFLECTIONS AT THE CENTER OF A UNIFORMLY LOADED, SIMPLY SUPPORTED, 30° SKEW PLATE FOR VARIOUS NETWORKS

Network	λ	Moment M_x	Moment M_y	Moment M_{xy}	Deflection w
Hexagonal........	$c/4$	0.143095 $p\,c^2$	0.160524 $p\,c^2$	− 0.015095 $p\,c^2$	0.041948 $p\,c^4/N$
Square............	$c/4$	0.144 $p\,c^2$	0.159 $p\,c^2$	− 0.0144 $p\,c^2$	0.0420 $p\,c^4/N$
Square............	$c/3$	0.12 $p\,c^2$	0.14 $p\,c^2$	− 0.016 $p\,c^2$	0.038 $p\,c^4/N$

center of the plate, the values there were obtained by means of a supplementary network which included the six points adjacent to the center and which permitted direct computation of the center values.

Prior to the more accurate computation (in general, to three significant figures), the writer made a rough calculation using only two significant figures and taking a network in which $\lambda = c/3$. In that instance, the span of the network in the direction of Y was 1.667 c; the skew, 31°; and the length of the irregular edge, 1.94 c. That such rough computations, which produce results very quickly, have engineering value is again evident from a comparison of the results given in Table 8.

JULIUS SIGMAN,[26] ESQ.[26a]—Various methods of analysis are used in this interesting study of skew plates. These methods involve solution by finite differences, by trigonometric series, and by power series. It is very evident that the author has done some painstaking research, and the results obtained seem to agree fairly closely.

When practical considerations are taken into account, it becomes apparent that, because of the limitations of the basic assumptions, skew slabs and structures present a vast field still to be explored. Such an exploration would help in connection with many problems that commonly occur in practice.

Five general methods of studying the structural action of concrete slabs and frames may be listed:

1. Experimental tests of prototypes;
2. Laboratory tests of large-scale models;
3. Model analyses (plaster models and photoelastic tests);
4. Semi-empirical analyses such as those developed by C. Bach,[27] and John R. Nichols;[28] and

[26] Brooklyn, N. Y.

[26a] Received by the Secretary January 2, 1946.

[27] "Elastizität und Festigkeit," 7 Auflage, Abschn, 9.

[28] "Two way Slabs in the Proposed Building Code for Boston and New England," by John R. Nichols, *Proceedings*, A.C.I., May-June, 1934, p. 504.

5. Theoretical analyses utilizing (*a*) differential equations, (*b*) difference equations, (*c*) "single-infinite" series of hyperbolic functions, (*d*) "doubles-infinite" Fourier series, (*e*) power series, (*f*) trigonometric series, (*g*) W. Ritz's [29,30] approximate energy method involving the principle of energy minimum, (*h*) Bessel [31] functions, and (*i*) short series of polynomials.

In ordinary theory, the plates are assumed to be uniform in thickness and the thickness is assumed to be small compared with the superficial dimensions. In the case of skewed reinforced concrete rigid frame bridges, the thickness of the deck cannot be considered small compared with the lateral dimensions; and the depth, furthermore, is variable so that ordinary theory does not apply.

In addition, the deflections and energy of deformation are assumed not to be affected by the vertical stresses. In the rigid frame bridge deck, deflections and strain energy are markedly affected by the vertical tensions, compressions, and shears.

The following problems with practical value, suggested for future investigation, are worthy of theoretical treatment as thorough as that presented by the author: (1) Skew plates with large deflections; (2) skew slabs with variable thickness; (3) continuous skew slabs; (4) effect of partial fixity of edges; (5) effect of curbs, sidewalks, and parapets on deck action in rigid frame bridges; (6) structural action at corners of skew slabs—a possible approach to the solution would be one using the theory of functions of complex variables; and (7) variable depth skewed slabs with curbs.

Also, since authorities do not agree as to proper procedure for skewed reinforced concrete rigid frame bridge analysis, a solution involving the theory of elasticity might accomplish much toward clarifying certain questions concerning which opinion differs.

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[29] *J. reine angew. Math.*, Vol. 135, 1908.

[30] *Ann. Physik* (4), Vol. 28, 1909, p. 737.

[31] "The Theory of Sound," by J. W. Strutt, Baron Rayleigh, Dover Publications, New York, N. Y., 2d Ed., 1945.

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DISCUSSIONS

FUTURE OF LAKE MEAD AND ELEPHANT BUTTE RESERVOIR

Discussion

By Edward Gerald Smith

Edward Gerald Smith,[110] Assoc. M. Am. Soc. C. E.[110a]—The analyses of the futures of two reservoirs, where the sedimentation problem is notably important and on which some of the most complete data on the subjects of sedimentation and closely related retrogression have been recorded, is a thought-provoking treatise, timely because of the proposals pending approval by Congress for complete development of several watersheds where similar problems are faced.

A learned analysis was made by the author to arrive at the conclusions on the probable life of Elephant Butte Reservoir. A historical reference is made to a news article published in 1916,[111] in which it was predicted that Elephant Butte Reservoir would have a service life of 233 years. The records used were computed by the late W. W. Follett, M. Am. Soc. C. E., covering the period from 1897 to 1912. Weight of sediment per cubic foot was determined by sampling started in March, 1916, of deposits formed by the high water of June and July, 1915. Beds for sampling were chosen at various locations to represent average conditions. The results from seventeen samples showed dry weights per cubic foot ranging from a maximum of 101.18 lb to a minimum of 87.90 lb. The average weight was 92.34 lb per cu ft. From these data was determined an average annual inflow into the reservoir of 11,336 acre-ft of sediment. Mr. Stevens' predictions, arrived at in a different fashion, indicate a reservoir life of 158 years, an annual rate of sediment inflow of 17,200 acre-ft, and an average specific weight of 65 lb per cu ft.

Note.—This paper by J. C. Stevens was published in May, 1945, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1945, by D. C. Bondurant, John H. Bliss, Luna B. Leopold, Carl B. Brown, and G. E. P. Smith; November, 1945, by Albert E. Coldwell, Walter B. Langbein, C. S. Howard, Charles Kirby Fox, H. V. Peterson, L. C. Crawford and P. C. Benedict, Charles P. Berkey, and Stafford C. Happ; December, 1945, by Berard J. Witzig; January, 1946, by Hugh Stevens Bell, E. W. Lane, and F. E. Bonner; and February, 1946, by Joe W. Johnson, Harold H. Munger, and William Mayo Venable.

[110] Asst. Engr., Hydrology Section, U. S. Engrs., Little Rock, Ark.

[110a] Received by the Secretary January 21, 1946.

[111] "Elephant Butte Reservoir to Serve 233 Years," by R. R. Coughlan and V. E. Lieb, *Engineering Record*, September 16, 1916.

This is not the best type of comparison but will serve to introduce the next thought concerning the accuracy of basic data. Sediment loads in rivers, as determined by past field methods, are of doubtful accuracy on all heavy sediment-laden rivers, primarily because such methods have been deficient in supplying data to obtain the true mean concentration of sediment in the flow cross section. It is doubtful if there are comprehensive data on any stream showing the distribution of sediment in the portion of flow near the stream bed which would include the saltation and bed loads. This information is especially lacking for the highest 20% of flows when 75% of the suspended sediment load is carried on large rivers increasing to more than 90% for smaller tributaries. Because of the tractive force required to begin bed-load movement the percentage of time and bed-load disparity is even more evident. This lack of information is due largely to field problems. Experiments made in late 1944 and 1945 on the Arkansas River at Van Buren, Ark., and Little Rock, Ark., with experimental devices for attempting to measure the bed load contradicted some important theories of bed-load movement and denied the practicality of using bed-load formulas. However, the experimental nature of the information precludes formation of definite opinions until further data have been obtained and studied. However, there is proof to indicate that significant quantities of gravel as large as from 1 in. to $2\frac{1}{2}$ in. are moved several feet above the stream bed even at moderate stages. With the Texas type sampler, which may sample much less than the true load at high concentrations, many results have been obtained showing sediment concentrations one foot above the bottom five times that at 8/10 depth and higher in the vertical. Similar data have been obtained by other departments on other rivers. These data emphasize the concentrated loads carried near the stream bed. To obtain accuracy comparable to that obtained in measuring river discharge, it is necessary to use procedures more involved and expansive than those required for obtaining discharge, since sediment load is a function of discharge and many other variables.

Sediment measurements, which are usually taken too infrequently for obtaining reasonably accurate definition of sediment hydrographs, and, concomitantly, sediment loads, must be taken much oftener than required for discharge measurements because of the greater inconsistencies in the character of sediment flow and because of the sharp irregular changes in sediment hydrographs in general.

Test data[112] show that none of the types of samplers tested is without serious deficiencies. Results showed sampling ratios of samplers compared to the depth and point integrations varying from 0.31 to 2.50. Errors are particularly large during periods of high flow and heavy concentration.

The writer draws the conclusion that, in addition to the possibility that field determinations of bed and saltation loads may some day require large additions to past records of sediment determinations to determine total sediment

[112] "A Study of Methods Used in Measurement and Analysis of Sediment Loads in Streams, Progress Report, Comparative Field Tests on Suspended Sediment Samplers December, 1944," planned and conducted jointly by TVA, Corps of Engrs., Dept. of Agriculture, U. S. Geological Survey, Bureau of Reclamation, Indian Service, and Iowa Inst. of Hydr. Research, published at St. Paul Dist. (Minn.) Sub-Office and Hydr. Laboratory, State Univ. of Iowa, Iowa City, Iowa.

loads, development of accurate samplers and improved field practices also may demand substantial increases to past records. All these findings suggest the possibility that adjustments for basic data being deficient in quantity load could result in a weight per cubic foot of dry sediment in the range of values determined by the 1916 field data.

Prolonging the Life of Reservoirs.—It is an obvious necessity that all feasible measures be adopted for prolonging the life of each project. The need for these measures is emphasized by noting that, in most reservoirs, the greatest quantity of sediment is deposited in the higher pools, especially in the first few generations, where the loss in storage causes a direct and immediate economic loss. In many reservoirs the amortization period may elapse before considerable deposits appear in the lowest portions of the storage. Reviewing the measures proposed for lengthening the life of Lake Mead with a view to applying lessons learned to other developments exposes great weaknesses in all proposals. Of the four measures proposed for prolonging the life of Lake Mead, three have been eliminated as possibilities for significant contribution to lengthening the useful life of a large reservoir. Removal of sediments after deposition is economically infeasible. It is unlikely that density currents in large reservoirs will ever result in carrying significant quantities of sediment. Then too, the material carried may be of the finer silts and clays which would deposit as "bottomset" beds in lower parts of reservoirs and handicap use of reservoir storage less. A large reservoir may be operated for two or more conflicting purposes. To attempt to operate the reservoir for an additional variable in order to utilize density currents for removal of sediment may be infeasible. Land management assumes a position of doubtful value when it appears that the sediment load could be reduced by no more than 10% by most careful applications of the principle. Then too, drought, an uncontrollable force of nature, may exercise more effective control over vegetation than a plan of land management can be expected to accomplish. In addition to the upstream storage, which will be discussed later, a program for bank stabilization would aid materially in preserving downstream reservoir storage. "Notes on Experiments on Meandering and Bed Load Movement"[113] emphasizes the importance of caving banks in conbributing to sediment loads in streams, especially of the larger grain-size sediments which deposit readily in pools. Stabilization of banks is especially necessary in the meandering reaches of river developments.

Upstream storage, as a reservoir storage protection measure, has many ramifications, possibilities, and flaws. In a longer range view, the construction of an upstream sediment-storage basin will lengthen the life of a particular reservoir. In a much longer range view both basins will be full of sediment and all resources in both areas permanently disabled. The point of view depends upon whether the engineer looks into the future in terms of decades, centuries, or larger units of time. In each valley development engineering studies for providing upstream storage involve difficulties in obtaining satisfactory sites close enough to the reservoir to have considerable effect, so

[113] "Notes on Experiments on Meandering and Bed Load Movement," Mississippi River Commission, U. S. Waterways Experiment Station, Vicksburg, Miss., May 1, 1945.

that intermediate sediment runoff and retrogression of beds by clear-water releases will not eliminate much of the effect of a sediment-storage project. Clear-water scouring channels will move the bed material, composed of larger grain-size material, which deposits readily in pools. It is necessary to evaluate the actual contribution to the life of the protected reservoir. In most instances, the sediment-storage project must be located so far upstream that benefits become small. Furthermore, the capacity of sediment-storage pools necessary to perform the required quality of sediment removal performance by detention of flows has yet to be determined within reasonable dimensions.

Benefits below sediment-storage pools are provided by retrogression of channels below dams providing greater channel capacity, and stabilization of channels accomplished by eliminating the supply of material, the latter being pointed out in model tests.[113] However, even these benefits are offset by increased flood and other land damages upstream from the pools caused by aggradation of sediments upstream from the deltas that are formed at the heads of reservoir pools.

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DISCUSSIONS

SEDIMENTATION AND THE DESIGN OF SETTLING TANKS

Discussion

By Rolf Eliassen

Rolf Eliassen,[50] Assoc. M. Am. Soc. C. E.[50a]—The expressed purpose of the paper is to develop the theory of sedimentation to such an extent that all the basic factors affecting the sedimentation of water and sewage solids may be so evaluated that it is possible to use mathematical functions to predict the economic design of settling tanks. Mr. Camp has developed mathematical functions for some of the variables affecting sedimentation. The questions that the designing engineer must ask are whether all the important factors have been considered, whether these factors have been evaluated properly, and whether the conclusions drawn are valid for conditions encountered in practice.

In the "Synopsis," the author makes a statement that "The settling characteristics of the suspensions to be clarified are rarely considered in the design of settling tanks." On the basis of his experimental work, Mr. Camp defines these settling characteristics in terms of overflow rates of from 200 gal per min to 2,000 gal per min per sq ft—which are equivalent to settling velocities of from 1.1 ft per hr to 11.1 ft per hr.

An analysis of the design of settling tanks selected at random from sewage treatment plants throughout the United States is shown in Table 3. These plants have been designed on the basis of overflow rates well within the limits prescribed by Mr. Camp. Some of them are close to the value of 667 gal per min per sq ft mentioned in Section 10 as a desirable overflow rate for the primary sedimentation of sewage. Thus, one would tend to believe that sanitary engineers, state boards of health, and equipment manufacturers have taken into account the settling characteristics of the suspensions involved.

The major difference between the practices of designing engineers and the conclusions of Mr. Camp is not in the matter of settling velocities and overflow

Note.—This paper by Thomas R. Camp was published in April, 1945, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1945, by Norval E. Anderson, and R. A. Mulholland; November, 1945, by P. Charles Stein, and Lynn Perry; and February, 1946, by E. Sherman Chase.

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[50a] Received by the Secretary January 31, 1946.

rates, but in detention times. Modern settling tanks in water treatment and sewage treatment plants are designed on the basis of settling periods of from 1 hour to 4 hours and involve the use of tanks with depths from 5 ft to 15 ft. The author comes to no definite conclusions as to settling times, since these are involved in mathematical functions which may be solved for each condition of flow and character of suspension. However, he does cite examples wherein detention periods of from 0.1 hour to 2.4 hours are calculated for tanks with depths ranging from 0.5 ft to 11.6 ft. Mr. Camp places particular emphasis on the need for tanks with short detention periods, shallow depths, and relatively high velocities of the liquids flowing through the tanks.

TABLE 3.—OVERFLOW RATES FOR SETTLING TANKS AT SEWAGE TREATMENT PLANTS

City	Type of treatment	Design flow (mgd)	Area of tanks (Sq ft)	Gallons per day per Square foot
Huntington Beach, Calif.	Trickling Filter	0.60	804	746
Sayreville, N. J.	Chemical precipitation	0.75	1,232	609
Birmingham, Ala. (Shades Valley)	Chemical precipitation	2.0	3,180	1,590
Dayton, Wash.	Trickling filter	3.6	1,964	545
Olean, N. Y.	Primary	5.0	3,927	786
Neenah-Menasha, Wis.	Primary	10.	8,836	1,132
New York, N. Y. (Coney Island)	Chemical precipitation	35	25,448	1,372
Atlanta, Ga. (Clayton)	Primary	42	39,408	1,065
Denver, Colo.	Chemical precipitation	54	61,576	880
Cleveland, Ohio (Easterly)	Activated sludge	123	157,632	780
Washington, D. C.	Primary	130	105,900	1,230
Buffalo, N. Y.	Primary	150	80,424	1,868
Chicago, Ill. (Southwest)	Activated sludge	400	395,000	1,010

Much attention is given to the derivation of equations involving the effect of velocity gradients on the flocculation of particles suspended in water and sewage. From these equations, the author proceeds to a consideration of the most economical settling tank design. Examples are presented leading to numerical values for tank dimensions. Some of the intermediate steps in the process should be recognized by the designing engineer so that he is cognizant of the limitations of this method of analysis.

In the first place, only the velocity gradients caused by drag on the floor and walls of a tank are considered by the author—the factors of inlet turbulence and density currents being neglected. Both of the latter may have an appreciable influence in tanks where the ratio of $\frac{L}{H}$ is comparatively low,[50b] in the range of from 2 to 10. These low ratios are found in most circular tanks and many of the shorter rectangular tanks in common use throughout the United States. Thus, any numerical or quantitative results derived from these mathematical equations must be examined critically to determine whether the basic assumptions made by the author are valid for the tank under consideration.

Within the limitations of long shallow tanks in which the drag on the tank floor and walls will have a dominating influence on velocity gradients, the author develops a sound theory leading to quantitative results in several

[50b] Correction for *Transactions:* In April, 1945, *Proceedings*, page 469, line 1, change "$\frac{H}{L}$" to "$\frac{L}{H}$"

examples. However, the suspensions given in the examples are hypothetical ones as far as velocity gradients and mixing coefficients are concerned. The author has specified the type of laboratory equipment that should be developed to obtain the necessary analyses of suspensions; but, as of early 1946, the equipment is not available. The logical approach to a correct analysis of the problem is by the construction and testing of pilot plants. Engineers frequently have difficulty in securing funds from their clients for pilot-plant work. For this reason, it is not easy to evaluate many of the factors influencing sedimentation. Therefore, designing engineers proceed on an empirical basis.

In Section 9 Mr. Camp departs from considerations of the theoretical aspects of sedimentation and discusses short-circuiting and stability in tanks. The writer believes that Mr. Camp places too much emphasis on short-circuiting as the criterion of settling tank performance. In the third paragraph of Section 9, the author states, "Short-circuiting studies are usually made on model tanks operating in accordance with Froude's law." This law is applied in an attempt to secure dynamic similarity between model and prototype when gravity forces predominate and leads to the equation known as Froude's number, $\left(\frac{V^2}{l\,g}\right)_p = \left(\frac{V^2}{l\,g}\right)_m$, in which the subscripts p and m refer to the prototype and the model, respectively. Gravity forces may predominate in the study of some settling tank inlets in the case where high velocities exist and head becomes a factor. However, Mr. Camp has chosen to neglect the effect of inlet turbulence and to consider only the case in which the forces of drag on the floor and walls of the tank predominate. These are viscous forces which also predominate in the settling of particles in water and sewage, as mentioned by Mr. Camp in Section 1. Consequently, if dynamic similarity is to exist with the predominance of viscous forces, the relationship between prototype and model should be governed by Reynolds' number, $\left(\frac{V\,l\,\rho}{\mu}\right)_p = \left(\frac{V\,l\,\rho}{\mu}\right)_m$. Mr. Camp presents no data based on model studies operated in accordance with Reynolds' law. Thus, the results as depicted in Fig. 15 are open to question.

Furthermore, the selection of an arbitrary hydraulic efficiency as the criterion of settling tank efficiency is not consistent with the theory previously developed in the earlier sections of the paper. In Section 4 Mr. Camp concludes that, in an ideal basin:

> "For any given discharge, the removal is a function of the surface area and is independent of the depth of the basin; or, the removal is a function of the overflow rate and, for a given discharge, is independent of the detention period * * *."

In Section 9 he states that:

> "A rough estimate of the effect of short-circuiting on removal may be had if it is assumed that the suspension is subjected to various settling times, distributed as indicated by the dispersion curve."

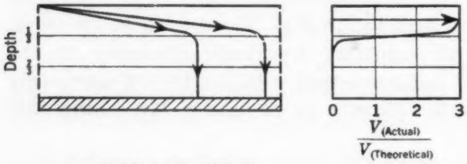
All the liquid passing through the tank is subjected to the same overflow rate, but various fractions of the liquid are given different detention times. If

overflow rate is the criterion, as claimed in Section 4, then no appreciable effect should be noted on the efficient removal of settleable solids in the tank. If detention time is the criterion, as suggested in Section 9, short-circuiting should have a marked effect. The former possible criterion is based on sound theory and the latter is based on an opinion, shared by many other engineers, but not borne out by experimental evidence.

Tests by the writer have shown that many circular settling tanks experience greater short-circuiting than do rectangular tanks; but the average effect on settling efficiency seems to be inappreciable. This statement is borne out by results in existing water treatment and sewage treatment plants. With similar overflow rates and theoretical detention periods, no great difference in settling efficiency is noted between the performance of rectangular and circular tanks at many plants throughout the United States. On the whole, average values of removals are quite consistently alike, as may be indicated by perusal of the wealth of operating data in the literature on the subject.[51] Selection of circular or rectangular settling tanks seems to be based on preferences of designing engineers and economy of plant design.

Consider a settling tank, either circular or rectangular, with a dispersion curve similar to curve B in Fig. 15. This has a "probable flowing through time," $\frac{t_A}{T}$, of 0.831. The author claims that the other curves shown in Fig. 15, each having higher values of $\frac{t_A}{T}$, and thus greater hydraulic efficiencies, "are characteristic of progressively better types of settling tanks."

For the sake of argument assume that the flow is concentrated in the upper third of the tank with the entire lower two thirds dead space. The average velocity will be three times the theoretical and the "flowing through time," $\frac{t_A}{T}$, will approximate 0.33. Since the lower part of the tank is assumed to be quiescent water, the paths of particles will be as shown in Fig. 21. On a hydraulic basis the tank would only be considered 33% efficient. However, a glance at Fig. 21, together with consideration of the significance of settling velocities and overflow rates, shows that particles will be removed as well as if the entire tank were utilized. This tank is merely the shallow high velocity tank recommended by Mr. Camp in Section 4 as the economical tank for settling.



(*a*) PATHS OF PARTICLES (*b*) VELOCITY DISTRIBUTION

FIG. 21

The main argument against this type of flow pattern might be that the time theoretically available for flocculation is not utilized to the fullest extent. However, this might not even be correct if the statement by Mr. Camp follow-

[51] "Sewage Treatment Works," by C. E. Keefer, McGraw-Hill Book Co., Inc., New York, N. Y., 1940, p. 125.

ing Eq. 31 holds true. He considers the effect of velocity gradients on flocculation and states,

> "From Eq. 31, it may be seen that the rate of flocculation due to velocity gradients is proportional to $V^{1.5}$ and inversely proportional to $\sqrt{R}$. Hence this type of coagulation is greatest for shallow tanks with relatively high velocities."

It must be admitted that short-circuiting can have an adverse effect on settling efficiency if strong currents travel along the bottom of the tank at such a rate that the tractive force will dislodge particles already settled, or will carry the particles along as part of the bed load. The effect of velocities on bed loads is discussed in Section 5. Inlet baffles should be so designed that this type of short-circuiting is avoided. As long as bottom velocities are low enough, short-circuiting in the upper regions of the tank should have no appreciable effect since overflow rate is the governing factor.

This discussion of the effect of short-circuiting is not meant to imply that no attempt should be made by designing engineers to secure good flow distributions in settling tanks. It is merely a plea for a rational approach to the determination of settling tank efficiencies. These should be determined by actual tank performance, as measured by solids removals, and not by an arbitrary assumption that settling efficiency is almost a direct function of hydraulic efficiency. By theoretical analysis in another section of his paper, Mr. Camp proves, and emphasizes time and again, that overflow rate is the controlling function in removal of solids. This is sound practice and should be the main criterion of settling tank design.

The conclusions in Section 10 do not constitute a summary of the salient factors developed from theoretical considerations in the main body of the paper. In his closing discussion Mr. Camp should summarize the most important conclusions regarding the clarification theory, the effects of tractive force and bed-load movement, and the effects of turbulence and flocculation on sedimentation. Such a summary would make the paper much more valuable to the designing engineer.

As presented, the conclusions are opinions of the author on the design of mechanisms and general arrangements for settling tanks. Several factors not mentioned under theoretical considerations and principles are introduced when the author abandons theory and writes about practice.

To utilize shallow tanks with high velocities, Mr. Camp suggests the design of a tray settling tank as shown in Fig. 17. Such tanks have been used for many years in the chemical and metallurgical industries for the sedimentation of particles somewhat different from those encountered in water treatment and sewage treatment plants. This type of tank has been made available to the sanitary engineering profession by a number of equipment manufacturers but has been accepted for relatively few installations.

In 1940 the writer saw one such unit installed in a sewage treatment plant at Springfield, Mo. Considerable difficulty had been experienced when the sludge that had settled on the trays began to digest. Entrapped gas caused some of the sludge to rise as scum and accumulate on the underside of the tray

above where it must have collected in considerable quantity and solidified to a degree. Further digestion of this scum released gas which could not escape readily. Pressure must have built up within the mass of scum and sludge to the extent that the concrete slab forming the tray was broken and lifted, with the virtual destruction of the tray. There is no assurance that the same condition might not develop in the design proposed by Mr. Camp. The mechanism would be required to scrape the sludge that rises as well as that which settles. Much experimental work must be conducted before a practical design can be developed. The design of final settling tanks for the activated sludge process does not necessarily result from the material developed in the paper.

In summarizing, it should be emphasized that Mr. Camp has accomplished one part of his announced objective—namely, "to collect in one compendium the known principles of sedimentation essential to the development of design theory." However, he has only partly accomplished his other goal, which he announced was "to present the theory of design developed to a stage which will permit its use in practice." Some of the numerical examples are based on hypothetical analyses, the laboratory equipment for whose determination is yet to be developed. Other factors influencing sedimentation are too complex for mathematical analyses or cannot be evaluated on the basis of the present state of knowledge of the art.

Mr. Camp's paper should be valuable to the designing engineer because it provides for a better understanding of the principles affecting sedimentation. It should also encourage pilot-plant studies of settling tank design, applying those principles developed in the paper. The accumulation of much data on the operation of shallow tanks using high velocities at appropriate overflow rates, together with the interpretation of these data by engineers and mathematicians, is necessary before many of the factors influencing sedimentation can be evaluated and formulated to such an extent that they are directly applicable to the problems of the designing engineer.

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DISCUSSIONS

CAVITATION IN HYDRAULIC STRUCTURES A SYMPOSIUM

Discussion

BY J. M. ROBERTSON, AND FRED W. BLAISDELL

J. M. ROBERTSON,[8] JUN. AM. SOC. C. E.[8a]—The theory of cavitation can be divided roughly into two basic parts: The first is concerned with the occurrence of cavities—that is, the conditions under which cavities form; and the second is concerned with the dynamics of the cavities once they are formed. Professor Vennard's treatment of these two phases is good although somewhat elementary. However, in being elementary, it may represent correctly the state of knowledge on the subject.

Under the heading, "Occurrence of Cavitation," it is stated that "* * * liquids encountered in engineering practice cannot expand and cannot support tension stress." Although this assumption is the basis for the commonly accepted analysis used to indicate when and where cavities will form, it is not an absolute fact. According to R. W. Boyle (65),[8b] some careful static experiments have proved that liquids can be placed under tension (see also mention of this possibility by T. C. Poulter (15)). This phenomenon is similar to that of the occurrence of supersaturated solutions. If sufficient disturbing conditions are present, the phenomenon will not occur. Also, the degree of tension which a liquid may stand without parting is probably a function of the time of application of the tension. Thus, for a very short time water may withstand a high tension although for a relatively long time it may be able to withstand very little tension. This time effect is of some significance with regard to the determination of the exact location in time and space at which a cavity will form. For example, in the flow through a nozzle, if a pressure determination indicates that negative pressures occur at the throat of the nozzle, the cavities will actually appear some distance downstream, the exact point of occurrence

NOTE.—This Symposium was published in September, 1945, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: December, 1945, by C. A. Mockmore; and February, 1946, by Joseph N. Bradley.

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[8a] Received by the Secretary January 21, 1946.

[8b] Numerals in parentheses, thus: (65), refer to corresponding items in the Bibliography, which appears as the last unit of the Symposium, and at the end of discussion in this issue.

depending upon the flow velocity. In the case of cavitation produced by vibratory methods, it is possible that, with a very high frequency of vibration, the part of the cycle during which the liquid is in tension would not be long enough for a cavity to form. The relation between the tension a liquid can stand and the time of application of the tension is also a function of the air or gas content of the liquid. Thus, in the case of the aforementioned nozzle, it is probable that, if the flowing liquid had a large dissolved gas content, the cavities would form much nearer to the point specified by elementary theory than they would if the same liquid contained little or no dissolved gas.

The theory of the action of a cavity, once it is formed, is of concern mainly in connection with the manner in which cavitation produces erosion in near-by solids. In addition to the work on the pressures caused by the collapse of cavities mentioned by Professor Vennard, reference should also be made to the theoretical analysis presented by E. H. Kennard (66) and the hypothesis advanced by F. D. Smith (67). Mr. Smith's hypothesis is that the liberated cavities or gas bubbles undergo resonant vibrations producing intense local strains in the vicinity and the destructive effects are due to these strains. In addition to the mechanical effect of the collapsing or vibrating cavity in producing cavitation erosion, it is possible that other phenomena resulting from cavitation may assist in the action. W. T. Richards (68) in his summary of the state of knowledge on the effects of cavitation due to sonic or ultrasonic vibrations noted that high temperatures have been shown to occur at the edge of cavities, that the collapse of cavities has resulted in the creation of electrical potentials, and that cavitation has been shown to accelerate chemical reactions. In his studies of phenomena due to ultrasonic vibrations in nonmetallic (liquid) systems, K. Sollner (69) concludes that all the disruptive and destructive phenomena are caused by cavitation. It is possible that such secondary effects of cavitation as these may have an effect on the manner in which cavitation erosion occurs.

The vibratory form of apparatus for producing cavitation has been used extensively for the rapid comparison of the cavitation erosion resistance of various materials by S. L. Kerr (35). This type of apparatus may also be used to further understanding of the action of the cavities. With it cavities are formed at a fixed frequency in a relatively restricted and known location. By high-speed photography it is possible to observe the formation, growth, and collapse of a cavity under known pressure conditions. Research, such as that of M. Kornfeld and L. Suvorov (70), along this line should yield information on the dynamical principles surrounding the life of cavities.

FRED W. BLAISDELL,[9] ASSOC. M. AM. SOC. C. E.[9a]—The Symposium authors have shown conclusively that the possibility of cavitation in hydraulic structures must be considered, not only by the designer but also by the constructor and the inspector. Several instances are cited where the cavitation can be traced to poor design. Other instances show that cavitation is also possible in well-designed structures as a result of irregularities built into the solid bound-

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[9a] Received by the Secretary February 6, 1946.

aries during construction. In fact, Mr. Harrold uses an assumed construction irregularity in his illustration of the "Principle of Vacuum Apparatus." All the authors have shown that extensive damage, costly and continuous repairs, and operating difficulties are the price paid for neglecting to consider cavitation. A conservative office design that would eliminate all possibility of cavitation might well be uneconomical. On the other hand, although a design based on laboratory tests can work close to the cavitation limit, the possibility of cavitation damage will be greatly reduced or eliminated. This has been implied by Professor Vennard under the heading, "Remedies for Pitting."

The writer well remembers the staccato hammering in the cavitation apparatus at the Massachusetts Institute of Technology in Cambridge that made conversation in its vicinity a practical impossibility. The writer does not doubt that the cavitation in the Norris sluices could be heard, as reported by Mr. Hickox. The writer would like to ask the authors if the noise level is related to the damage caused by collapse of the vapor pocket.

At the end of the third paragraph under the heading, "Damage from Cavitation, or Pitting," Professor Vennard writes,

> "In some cases, however, the pitting has stopped of itself, apparently due to a water cushion covering the eroded region and preventing direct contact of collapse point and solid material."

The writer would like to inquire if it is desirable in some instances to design for the elimination of cavitation pitting rather than for the elimination of cavitation itself. Such a design would incorporate a "water cushion" into the structure to absorb or distribute the forces resulting from the collapse of the vapor pocket rather than streamlining to eliminate cavitation. Under the heading, "Proper Use of Baffle Piers," Mr. Harrold recommends the streamlining of baffle piers to eliminate cavitation where velocities are high and static pressures are low. Streamlining requires careful form work and excellent workmanship. Mr. Warnock mentions the extensive damage in the Boulder Dam spillway tunnel resulting from a misalinement in the tunnel invert, and Mr. Hickox shows that cavitation pitting was caused by misalinement of joints in the sluice liners at Norris Dam. These experiences show that "streamlined" baffle piers would have to be constructed carefully if all cavitation were to be eliminated. On the other hand, if the piers were deliberately made narrower in a downstream direction, the vapor pockets would collapse in the liquid. Careful form work would not be required and pitting of the pier would presumably not occur. This is in line with Mr. Harrold's statement in the last paragraph under the heading, "Baffle Pier Tests," that "A cavitation streamer that leaves the baffle is assumed to be harmless, whereas one that clings to the baffle is considered harmful." On the other hand, the writer would like to ask if the pressure waves set up by the collapse of the vapor pockets might cause vibrations that would eventually weaken the pier. This important question has already been raised by Professor Vennard in the last paragraph under the heading, "Collapse of the Cavity." Perhaps he or Mr. Harrold will be willing to comment more fully in their closing discussions on the points raised in this paragraph.

The design of "water cushions" into a baffle pier might (in the words of Professor Vennard under the heading, "Remedies for Pitting") "have little appeal," but the pier would probably be cheaper to construct than if it were streamlined. The "cushioned" piers would possibly be more effective energy dissipators than would the streamlined piers. Before this method could be applied to the design of piers, it would be necessary to know how close the cavitation streamer must be to the structure in order to cause pitting (mentioned by Mr. Harrold), the location of the cavitation streamer, and how far downstream the vapor pockets travel before collapsing. It is quite apparent that much experimental work remains to be done before adequate information on cavitation and its effects will be available.

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DISCUSSIONS

DESIGN DEVELOPMENTS—STRUCTURES OF THE TENNESSEE VALLEY AUTHORITY A SYMPOSIUM

Discussion

BY I. L. TYLER, AND PHILIP SPORN

I. L. TYLER,[15] M. AM. SOC. C. E.[15a]—The field embraced by Mr. Riegel's excellent paper in this Symposium is such a large one that only a very brief treatment of many important features could be included. In the case of "foundation restraint" (see heading, "Concrete Dams: Foundation Restraint") some amplification of the text may be desirable.

In addition to the three major methods for controlling mass concrete temperatures discussed by Mr. Riegel another method was used at Hiwassee Dam. In this case, the mixing water for concrete was artificially cooled during periods of high placing temperature. Although the reduction in maximum temperature accomplished by this means was not much (perhaps 10% of the maximum rise), its possible effect on cracking cannot be ignored, since conditions favorable to the elimination of temperature cracks were actually closely approached and even small improvements in temperature conditions may have had considerable effect. Hiwassee Dam as finally completed was unusually free from cracks considering the very limited use of embedded cooling pipes. Perhaps some credit for this must be assigned to the benefits, even though small, which resulted at critical times from the cooling of the mixing water.

Mr. Riegel's discussion of the effect of the concrete placing rate on temperature rise of mass concrete without cooling pipes may not be sufficiently detailed to clarify all the variables involved. With a concrete placing rate averaging 1 ft per day the use of $2\frac{1}{2}$-ft lifts at $2\frac{1}{2}$-day intervals in place of the more common 5-ft lifts at 5-day intervals does permit some greater head loss to the air, as Mr. Riegel states, even with the low-heat cement and concrete of low-cement content. However, an even greater advantage, so far as cracking from founda-

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[15a] Received by the Secretary January 5, 1946.

tion restraint is concerned, comes from the improved temperature distribution vertically through foundation and dam. The desirable condition, of course, is to avoid steep temperature gradients and for this the thin lifts are preferable as shown by R. W. Carlson,[16] Assoc. M. Am. Soc. C. E.

Mr. Riegel made no comments on the effectiveness of the measures taken to minimize temperature cracking in mass concrete. This may leave the reader with some doubts as to the feasibility of reducing cracking by such precautions. It is realized that many factors other than temperatures influence cracking in mass concrete, and that to cover them all would be out of the question. However, the writer suspects that Mr. Riegel has a considerable fund of information on the relation of temperatures and cracking on many of the TVA structures, and he respectfully urges that Mr. Riegel give the matter all consideration possible in his closure.

PHILIP SPORN,[17] M. AM. SOC. C. E.[17a]—As stated by Mr. Meyer, the civil engineering features of the design of any steam plant materially affect the ultimate cost of power generation. In the writer's experience, this has been amply demonstrated many times. However, the writer is of the further opinion that the design of both the electrical and mechanical systems, so far as arrangement of equipment and the selection of materials and methods for doing the work are concerned, are just as flexible and present just as many possibilities for savings in the initial cost of the development as do the civil engineering features. Therefore, all three phases should be considered at the same time and only when this is done and the selection of the best combination is made does the most economical design result.

Furthermore, the operation and maintenance of the plant must be given careful consideration, or the savings realized from some particular design may be quickly wiped out. This is particularly important when the present cost of labor and the trend that seems to be establishing itself in this field are considered.

The advantages of economical fuel supply, and the location with regard to load, must have been the main determinants of the location of the Watts Bar Steam Station because the other reasons given by Mr. Meyer for choosing this site are not very conclusive. The savings from combining the construction of the hydro plant and of the steam plant are small; and, from an operating standpoint, the plants are too far apart to reduce the size of the operating personnel appreciably. Possibly, if the hydro plant were electrically controlled from the steam plant, some saving in operating costs could be realized but Mr. Meyer does not mention if this was done.

The design developed by Mr. Meyer is well prepared and rationally developed. Rationalized as it is, however, some of the underlying assumptions seem open to question. At least, they run counter to the writer's experience. To illustrate: At Watts Bar, two stacks have been provided, each probably

[16] "Temperature and Stresses in Mass Concrete," by R. W. Carlson, *Journal*, A.C.I., March-April 1938.

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serving two units. The writer has found that the cost of this type of arrangement and the amount of critical materials employed are about the same as in the case where a separate stack is provided for each boiler. From an operating standpoint the individual stack for each boiler is preferable to the arrangement adopted at Watts Bar.

From a study of the plans, the width of the generator room seems to have been determined by turbine erection, considering that the generator rotor would be pulled parallel to the long axis of the machine. In pulling the rotor, if it were canted in a horizontal plane (as is frequently done), the width of the generator room could have been reduced. Furthermore, the writer has never found any need for carrying the track approximately the full length of the turbine room as has been done at Watts Bar. Generally, this increases the width of the generator room.

Furthermore, a wall seems to have been constructed between the generator room and the heater bay. The trend in modern design omits this wall. By opening the generator room to the heater bay and boiler room it is possible to obtain not only savings in structures but better ventilation. The operating personnel avoid the feeling that the boiler room is a place for burning coal and as a result must be dirty and does not deserve the same clean care as the generator room.

Since the main basement floor is only 19 ft below the level of maximum river stage, it is difficult to understand why the foundation slabs need to be as thick as indicated in Fig. 17. For example, at the Philo Plant of The Ohio Power Company, where the bottom of the condenser pit is about 56 ft (as compared to 32 ft at Watts Bar) below ground level, and 46 ft (as compared to 19 ft at Watts Bar) below flood level, the slab was about 4.5 ft thick as compared to a slab thickness of about 8 ft at Watts Bar. Furthermore, it has been the writer's experience that, wherever there is a deep basement, it usually has been possible to arrange columns and struts so that one wall of the pit was braced against the other. As a result, the pit side walls at the Philo plant are about 3.5 ft thick as compared to the apparent 11.5-ft south wall of the boiler room basement at Watts Bar. It would be interesting to learn more about the design basis behind these differences.

Mr. Meyer states that the slab beneath the main boiler columns was continuous for 250 ft and that the design of coal-bunker framing could not allow any movement of the foundations. Obviously, if the sandy shale foundation rock would only permit a loading as light as was applied, then some, perhaps substantial, saving could have been obtained by arranging the framing between the basement and main floors as a truss and reducing the thickness of the slabs. Was this considered and, if it was, and then rejected, what was the reason?

The handling of the circulating water problem and the adoption of the gravity system of supply is very interesting. From Fig. 19 the 3,500 ft of intake, which required such a deep excavation, appears quite costly, especially since the two pipes were not run together—apparently requiring 7,000 ft of excavation. Is such an intake required for the gravity system as economical and reliable as the use of circulating water pumps, considering that any settle-

ment might break the lines? Was any procedure in addition to the tongue on each length adopted to prevent relative movement between adjacent pipes because of earth movements? Why was such a long canal used to connect the condenser outlet with the river? A long canal permits more sedimentation than would a short one, and this has an adverse effect on the operating expense, as well as on the initial investment. Again, if the length of the canal had been decreased, the power plant could have been nearer the river—thus decreasing the length of the conveyer between the barge-unloading dock and the hopper building, which would have resulted in a saving in the cost of constructing the coal-handling facilities. Could Mr. Meyer enlarge on this subject?

The facilities that have been provided in the coal-handling yard are unquestionably ample; and, although they may be too elaborate for present capacity, they appear adequate for the ultimate capacity. Nevertheless, one is inclined to ask the reason for the choice of a drag scraper with a traveling tail tower, instead of large tractor-scraper equipment. One is also inclined to ask some questions about the coal dock, although there is not enough description included in the paper to comment on its construction. The Cabin Creek Plant of Appalachian Electric Power Company, with an installed capacity of 325,000 kw, has a sheet-pile dock about 200 ft long supplemented by pile clusters for mooring downstream from the dock. This arrangement proved to be very economical and yet substantial.

Mr. Meyer and his architectural associates are to be complimented for the fine synthesis of structural function with architectural appearance that has been incorporated into the Watts Bar plant. The writer is acquainted with no modern power plant at which this has been done more effectively and where the two functions have been more perfectly wedded. Engineer and architect have not only succeeded in developing a structure that functions properly, but have also obtained one that looks well.